PART 2
TEST FILLS

Chapter 8
General Considerations

8-1. Background

The earliest rockfill dams in the U.S. were built in the southwest and west just before the turn of the century (Wegmann 1899). Most were of loosely dumped quarried rock with some version of core or upstream facing including wooden planking, concrete, or hand-placed rock dry-wall. From thence up until the 1950's, the design and construction of rockfill dams were a matter of empiricism. Construction was by end-dumping over high slopes with water sluicing in 18- to 61- m (60- to 200-ft) lifts. The sluicing with water jets was intended to displace fines from between the larger particles to produce rock-to-rock contact among the larger particles and reduce the compressibility of the mass. However, the technique still produced rockfill which was relatively compressible and subject to considerable post-construction volume change. The transition to compacted rockfill for both earth-core and concrete-face dams occurred during the period 1955-1965 (Cooke 1984) as shown in Figure 8-1 (Cooke 1990). This transition was possible because of the advent of heavy vibratory rollers and was particularly spurred by Terzaghi's criticism of dumped rockfill for its excessive compressibility and his recommendation of compacted rockfill in thin lifts as a means of greatly reducing it and also allowing the use of poorer quality rock (Cooke 1960). In the United States, the 136-m-high (445-ft), Corps of Engineers Cougar Dam (completed in 1964) was the first major earth and rockfill structure in which vibratory rollers were used to compact the rock shells (Bertram 1973). At the time of the construction of Cougar Dam there existed practically no information about the construction and evaluation of compacted rockfill so that trial and error test-fill procedures were used as the work progressed. It is interesting to note that Terzaghi had stated earlier that it would be impossible to determine the properties of rockfill in the laboratory and that only experimental fills should be used for such purposes. Even the most recent literature (NATO 1991), though filled with laboratory and model study information on rockfill properties and behavior, still confirms a continued reliance on test fills. Notwithstanding that statement, it can also be said that, in overview of the significant experience gained and common current practices concerning rockfill, test fills may sometimes be in wider use than actually necessary. In the remaining portion of this Part of the manual, test fill will be spoken of in the singular but it is not at all uncommon that more than one test fill may be needed. The reader should have no difficulty in recognizing the aspects of that to follow which may dictate more than one fill.

8-2. Why a Test Fill?

The main properties of interest of compacted rockfill fall under or relate to, shear strength, compressibility, permeability, and suitability of compaction equipment. Because of the fundamental nature of rockfill being cohesionless and containing large particles, it is not feasible, nor is it possible to obtain or test large "undisturbed" samples to determine the pertinent properties. Furthermore, the typical three-dimensional heterogeneity of rockfill and the densities typically obtained from field compaction cannot be replicated in reconstituted laboratory specimens in those limited cases where very large laboratory testing equipment of high load capacity is available. Laboratory studies of rockfill properties have been conducted on gradations containing smaller maximum particle sizes than most often actually placed and have, therefore, been more akin to parameter studies to provide insights on effects of variations in those parameters and to provide educated estimates of full-scale gradation behavior. In specific case histories, such data can be applied in numerical analyses coupled with observed embankment behavior to assess the quality of the laboratory results for predictive purposes but the state of that art should probably be considered to be in a state of relative infancy. Even the more frequently performed versions of maximum density tests have usually involved altered gradations (scalped) or modelled gradations (scalped/replaced or parallel) with significantly smaller maximum particle sizes. The profession has not thoroughly established the effects of such practices on the numbers yielded in comparison with full-scale materials. Test fills have then often been the basis for determining traits of the compacted rock which have led to completely satisfactory dam embankments including the very highest yet constructed. If the rock is of high compressive strength (sound rock), test fills may not even be necessary or adequate placement and compaction procedures can be determined in the early stage of construction without elaborate test fill operations. In this case, the only tests needed are drill core samples and saturated unconfined compressive tests which are among those previously mentioned in Part 1. Cooke further states that for sound rock, four passes of a 9.1-Mg (10-ton) vibratory roller upon layer thicknesses averaging about 1 m (3.3 ft) have become standard practices. Heavier rollers have not been found to usually offer any advantages. Since permissible maximum particle size for sound rock can be equal to the lift thickness if the proper placement method is used (to be discussed later), the most efficient
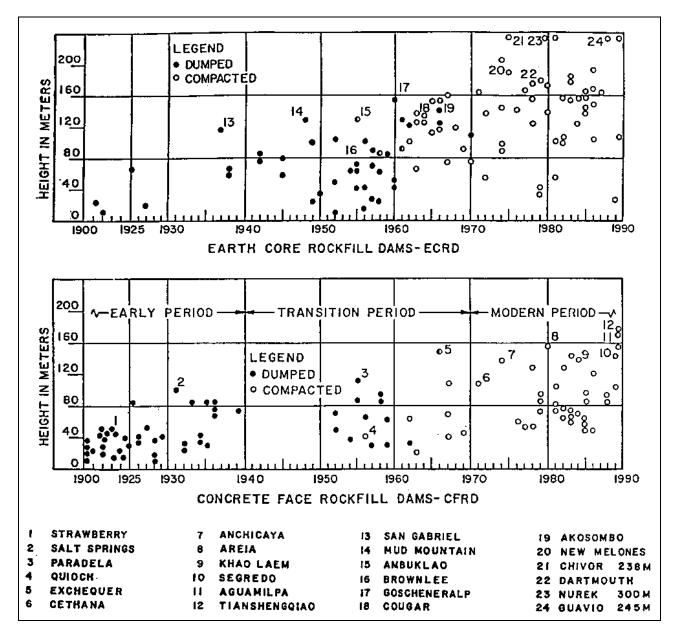
Figure 8-1. Transition in practice from dumped rockfill to compacted rockfill (after Cooke 1990)

quarrying operations determined from the test quarry may essentially dictate the lift thickness. If the available rock material is of low compressive strength (say, less than 55×10^6 Pa or 8000 psi), a test fill program is typically necessary. It has been previously stated in Part 1 that for softer rock types or conditions, degradation of the material from the quarry through all aspects of its handling including loading, processing (if employed), hauling, stockpiling (if employed), placement, and compaction (whatever the combinations of lift and equipment) cannot be confidently predicted by even the most experienced individuals much less the best placement/compaction

procedures. Indeed, the question sometimes exists as to whether the material will ultimately be a free-draining rockfill after compaction or whether it will have degraded or must be made to degrade (because it will do so eventually postconstruction) into a soil material and treated as such in all aspects of design, construction, and construction control. An example of material which may appear to be a rock upon quarrying but will deteriorate into a soil upon wetting (whether stockpiled or compacted in the embankment) with time are certain shales (Lutton 1977). In planning and conducting a test fill program, it should be kept in mind that it can also offer considerable

advantages in optimizing design and providing project construction personnel with the opportunity to familiarize themselves with materials and construction procedures.

8-3. Representative Procedures

A most important consideration for any test fill program is that procedures employed in constructing the test fill must simulate, as closely as possible, feasible construction procedures to be used in the project fill. The achievement of this imperative objective requires some experience in the construction of rockfill. If test-fill procedures do not closely simulate actual construction, the value of the test-fill investment is compromised and the effort may even do more harm than good. If experience in rockfill construction and its sampling/testing is seriously lacking, the use of a test fill as a preconstruction training exercise for project personnel may be a justified investment for sound rock and a natural advantage of test fill programs usually required for softer materials.

8-4. Test-Fill Scheduling

It has been by far the greatest preference to conduct test fills before construction begins (i.e., at some time during the project design stage) but there have also been cases of provisions made in the bid documents to allow for their construction during the early phases of actual construction. If the latter approach is under serious consideration, it must be based on very substantial confidence that the items to be determined from the test fill have no potential of altering the design of the embankment or of rejecting the basic adequacy of the available materials. On the other hand, the advantages of a prebid test fill include: results can be used by the designer to prepare specifications for rock placement and compaction (and blasting/processing if a test quarry is also conducted), the quarry face can be inspected by prospective bidders, and construction personnel can be trained for adequate visual observation skills and required testing procedures. Therefore, a properly conducted prebid test fill program will most likely result in a lower bid. A prebid fill would naturally be scheduled to start at a point in the iterative-step development of the test quarry such that gradations produced in the test quarry and available for the test fill construction are deemed to be those recommended for project construction. The decision of when to conduct a test fill, then, is one which must be based on features of the individual project.

8-5. Flexibility

A test fill program must be flexible. Because of natural rock variations and unpredictable behavioral characteristics, it is often impossible to lay out a definite program in advance from which there will be no deviations. Procedures and envisioned specifications have often been altered based on results of completed portions of an original program. The test fill program designers must anticipate that possibility.

Chapter 9
Planning and Design

9-1. General

Planning and design of a test fill program should be done with care to consider all the facets of the objectives of such a typically expensive investment. The proposed program should be thoroughly reviewed to assure that all procedures and tests are properly designated and planned in the order of the work. There is no better guidance available for laying out a program than to review those programs conducted by others, particularly for Corps of Engineer projects. This manual cannot substitute for the careful review of the details of procedures and findings to be found in the reports of test fills and test quarries for previous projects. As was suggested for a test quarry program, it is highly desirable that one individual be charged with responsibility in the field for conduct of the test fill and for dealings with the contractor.

9-2. Location of the Test Fill

The test fill should be located as near the test quarry or rock source as possible. This will obviously provide an economy of operation. If multiple test quarries are to be developed and multiple test fills associated with their yields, the siting considerations include the decision of multiple test-fill sites or a single larger site. The use of a stockpile between the test quarry and test fill operations depends upon the expected project construction operations. It has already been pointed out that stockpiling may produce changes in the rock gradations reaching the fill if for no other reason than the double-handling (loading and hauling). If stockpiling is not anticipated in the project construction, it should be avoided in the test-fill program if possible. If stockpiling is expected to be required in project construction, its effects should be assessed in the test-fill program. The test-fill site should be as level and of sufficient area to accommodate the test fill itself plus ample peripheral space to permit full equipment mobility. The site should be graded to provide good drainage.

9-3. Geometry

The geometry of the test fill configuration depends on the objectives and the variability and availability of the rock to be tested, not to mention constraints imposed by cost. In addition, there is considerable latitude deriving from individual preferences. It then becomes practical herein to only discuss test-fill geometry in the more general

sense. The test fill should be of sufficient size to allow its performance to be as close to project fill behavior as possible. This means that the effects of scale should be minimized. Widths and lengths of individual test sections should be of sufficient magnitude so that settlement readings (discussed later) reflect densification from compactive effort alone and do not reflect lateral bulging of the fill. In most cases, a width of 10 to 15 m (about 30 to 50 ft) with a length at least equal to the width but 6 to 10 m (about 20 to 30 ft) longer, if feasible. The individual fill sections may abut each other longitudinally or be layed out in a parallel configuration with ramps on each end at slopes of 1 vertical on 5 horizontal or flatter to facilitate equipment entrance and exit. Maximum side slopes of 1 vertical to 1.5 horizontal are recommended. The ramps and side slopes may be constructed of quarry-run materials. Four or five layers (lifts) are usually sufficient to provide enough data to establish the compaction specifications for any one type of rock. Figures 9-1 through 9-10 provide examples of test fill geometries used for several Corps of Engineers dam projects.

9-4. Test Sections or Lanes

In the most ideal case allowing the easiest separation of variables, an individual test section or lane of a test fill should not contain different materials or be composed of different lift thicknesses, lifts compacted by different equipment, or a different number of passes applied to succeeding lifts. For example, suppose it is desired to evaluate 46- and 91-cm (18- and 36-in.) lifts. It would be more desirable to use two fill sections, one containing 46-cm (18-in.) lifts only and the other 91-cm (36-in.) lifts only, rather than one section containing lifts of both thicknesses (e.g., four to five 46-cm (18-in.) lifts over four or five 91-cm (36-in.) lifts). There have been cases where groups of different lift thickness were employed successfully in the same section with increasing lift thickness from bottom to top of the section as shown for Seven Oaks Dam in Figure 9-10. The use of the transition from thinner to thicker lifts from the base upward at least diminishes the effects of additional settlement of the lower lifts being included in with the measurements assessing the compaction applied to the upper, thicker ones. However, the Seven Oaks test program heavily relied on large-scale density tests taken in each lift after intermediate roller passes to assess compaction rather than strictly relying on surface settlement readings. In other cases of a single test section incorporating more than one lift thickness, measures were taken to eliminate the continued settlements of lower, thinner lifts from entering into the settlement readings for the upper lifts. This was
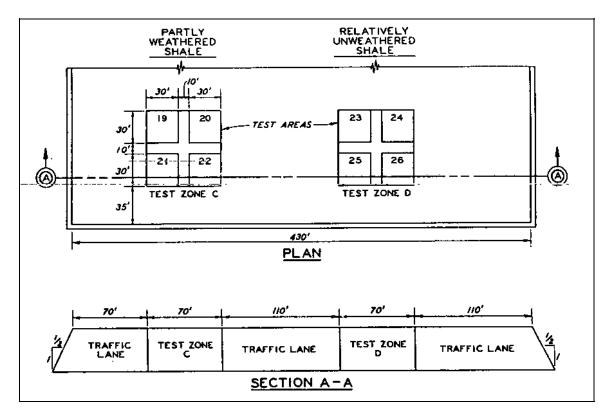
Figure 9-1. Beltzville Dam, plan and profile of the test fill

accomplished by either (a) rolling the last lower and thinner lift until no further settlement was seen before placing the upper, thicker lifts, or (b) installing settlement plates on the surface of the last, lower, thinner lift in order to subtract additional settlement from that observed for the upper lifts. The former method of "proof" rolling the lower lifts may completely alter that material with respect to its condition after a reasonable number of roller passes to be used in the project and compromise any observations from an inspection trench excavated after completion of the test fill. The use of settlement plates with stems up through additional thicker lifts poses troublesome obstructions in the placement and rolling of those lifts and may compromise their similarity to project conditions. In any case, enough lifts (four or five) of the same thickness must be used so that a good average settlement curve can be obtained for all like lifts in each zone.

9-5. Equipment

Generally, loading and hauling equipment should be used that will result in the most efficient operation and which is likely to be used for the project construction.

Front-end loaders can be used to load the quarried rock into trucks for hauling to a processor or to the test fill. A loader is more maneuverable than a power shovel and less costly on small operations. Crawler tractors are the standard equipment for spreading materials to the desired loose lift thickness and in many cases of medium to soft rock have also proven capable of breaking down oversized pieces delivered to the test fill. In special cases (i.e., not very frequently), where crawler tractors have been seen to produce excessive degrading of the material, rubber-tried equipment has been used for spreading. For material which does not degrade through the compaction operation to the extent that it must be considered a soil, 9.1-Mg (10-ton) or 13.6-Mg (15-ton) vibratory rollers are the most common choices. For materials which are friable or weathered material which will degrade into an obvious soil during hauling, placement, and compaction, heavier vibratory, pneumatic, or tamping rollers may be required. For materials which arrive at the test fill or are broken down in spreading and compaction into a mixture of rock and soil, the means of determining whether they remain suitable for rockfill or must be treated as a soil in design, construction, and construction control will be addressed in Chapter 10.
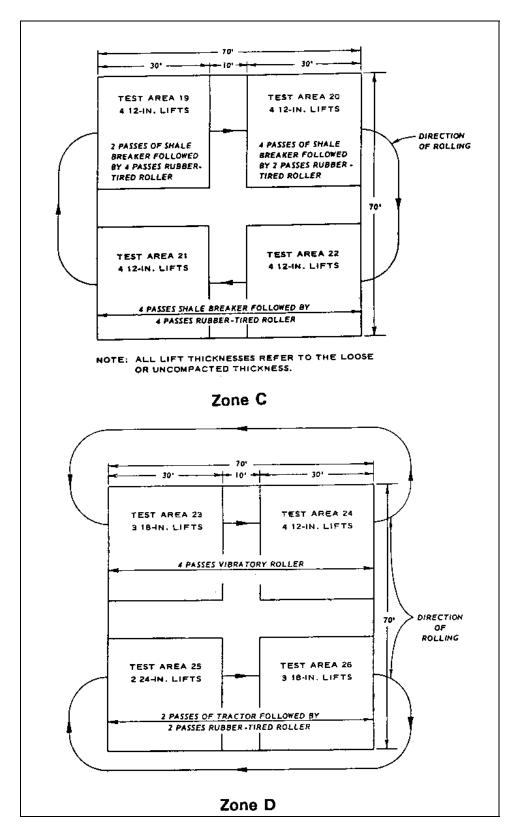
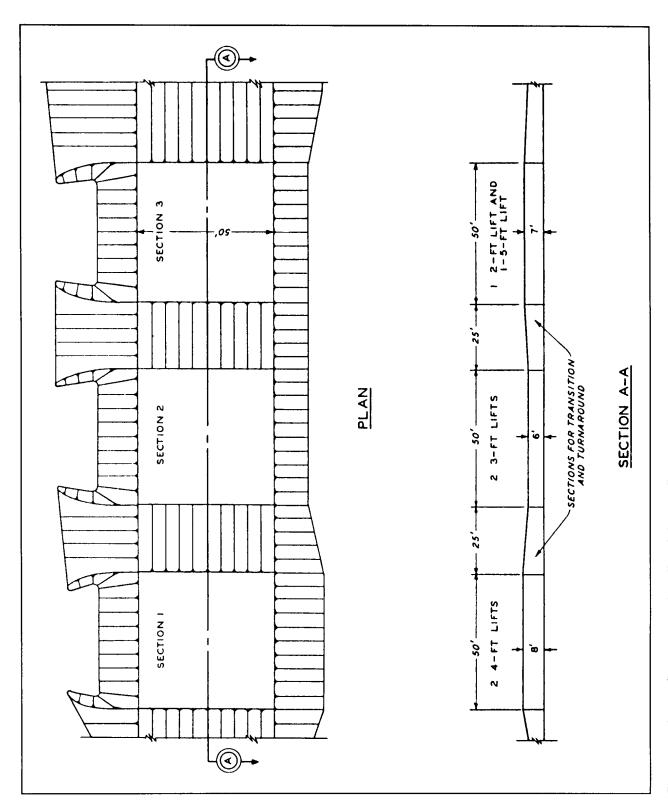**Figure 9-2. Beltzville Dam, plan view of the rolling pattern**

PLAN

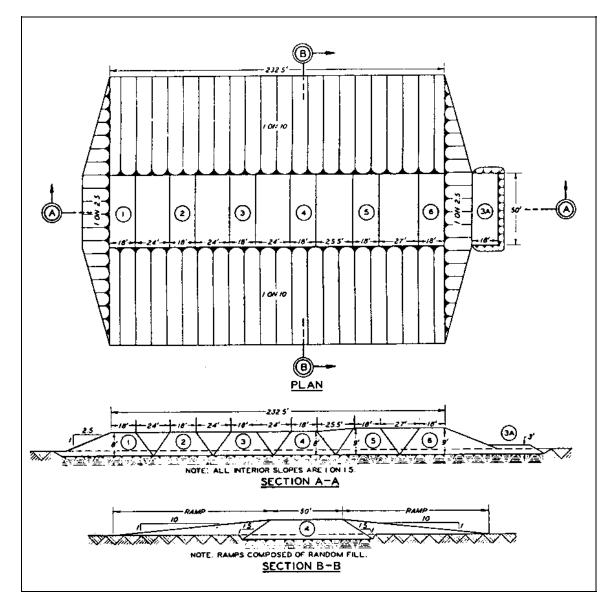SECTION A–A

Figure 9-3. Laurel Dam, plan and profile of the test fill

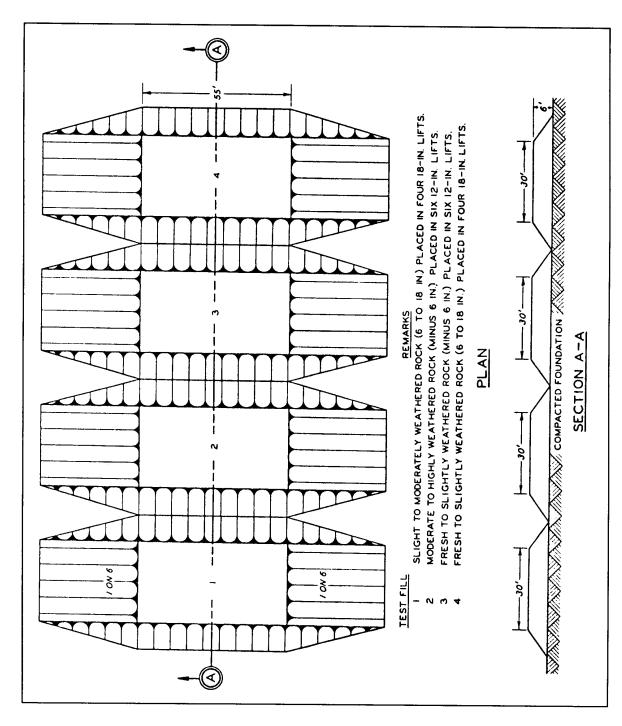**Figure 9-4. Gilham Dam, plan and profiles of the test fill**

TEST FILL

REMARKS

1   SLIGHT TO MODERATELY WEATHERED ROCK (6 TO 18 IN.) PLACED IN FOUR 18-IN. LIFTS.

2   MODERATE TO HIGHLY WEATHERED ROCK (MINUS 6 IN.) PLACED IN SIX 12-IN. LIFTS.

3   FRESH TO SLIGHTLY WEATHERED ROCK (MINUS 6 IN.) PLACED IN SIX 12-IN. LIFTS.

4   FRESH TO SLIGHTLY WEATHERED ROCK (6 TO 18 IN.) PLACED IN FOUR 18-IN. LIFTS.

PLAN

COMPACTED FOUNDATION

SECTION A-A

Figure 9-5. New Melones Dam, plan and profile of the test fill
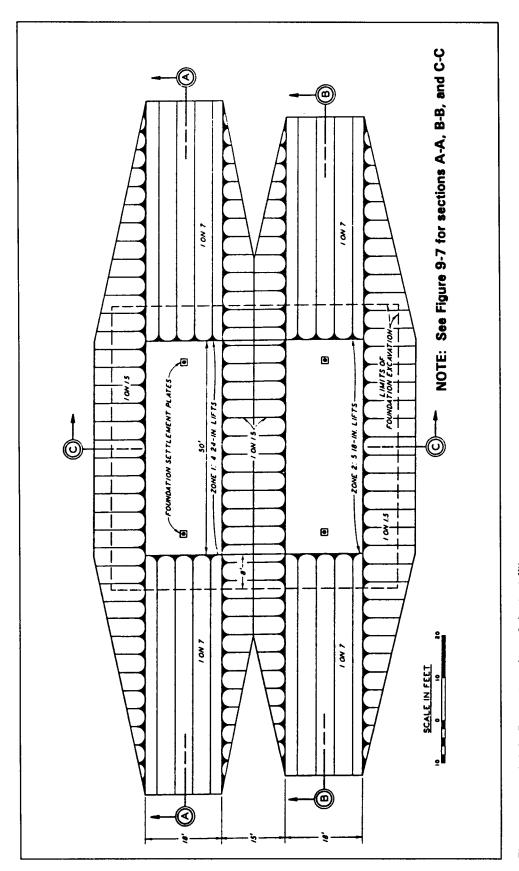
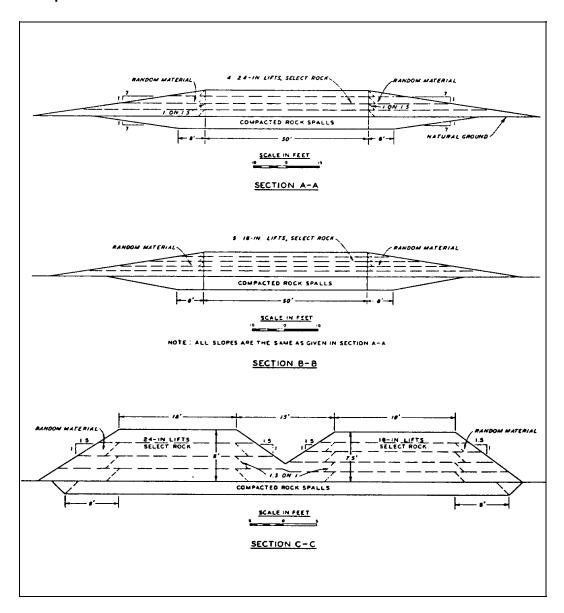Figure 9-6. Gathright Dam, plan view of the test fill

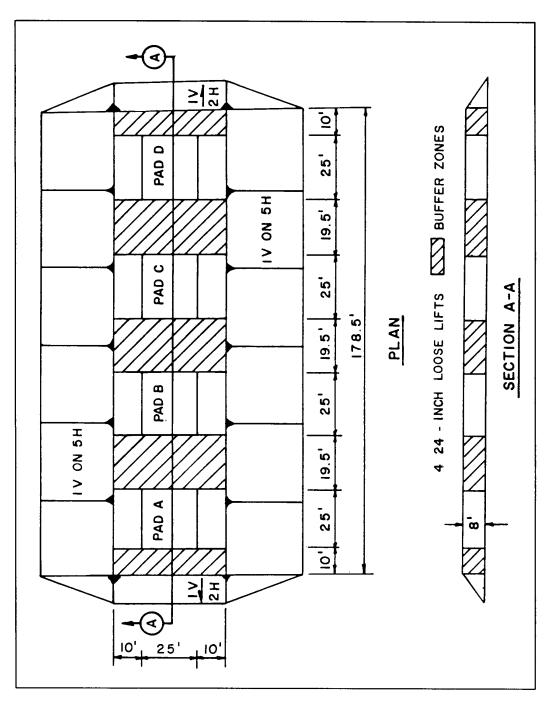Figure 9-7. Gathright Dam, profiles of the test fills

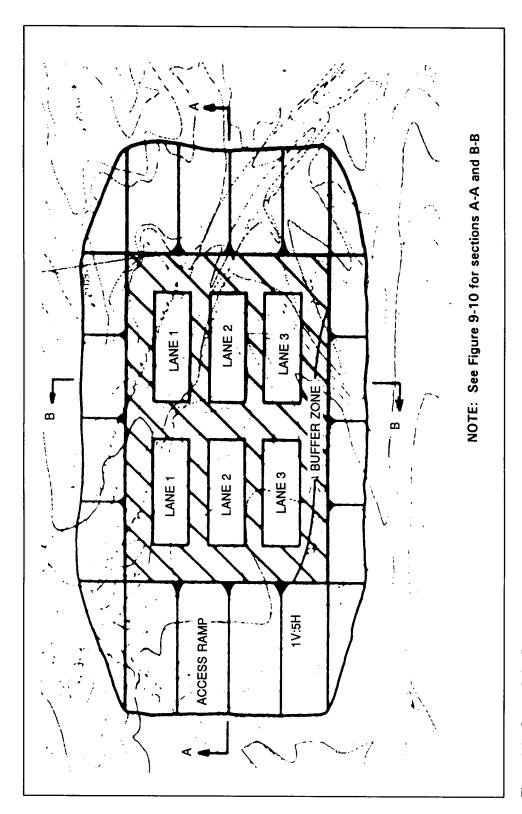Figure 9-8. Cerillos Dam, plan and profile of test fill No. 3

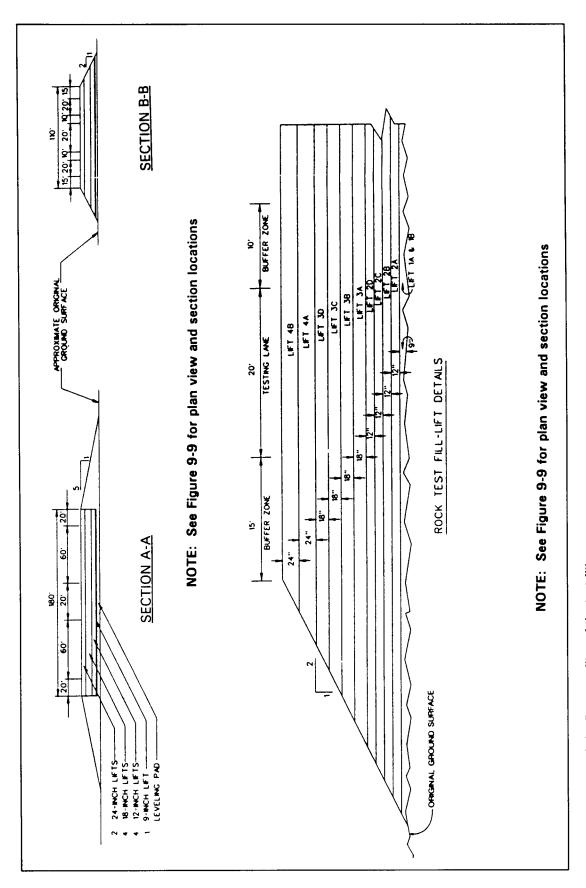**Figure 9-9. Seven Oaks Dam, plan view of the test fill**

NOTE: See Figure 9-10 for sections A-A and B-B

Figure 9-10. Seven Oaks Dam, profiles of the test fill

Chapter 10
Test Fill Construction

10-1. Foundation Preparation

The proper preparation of the foundation for a test fill is of special importance since settlement readings on the surface of the lifts rather than in situ density tests are commonly used to evaluate the relative compaction obtained. Fortunately, in areas near quarry sites, rock foundations can usually be provided with a minimum of overburden stripping. If, however, the foundation consists of soil or weathered rock, it must be thoroughly compacted prior to fill placement, preferably until no further significant settlement can be observed. Although undesirable, where further consolidation of a compressible foundation under fill loads is possible, settlement plates should be installed in the foundation to provide data needed to correct the test fill settlement readings. If foundation settlement plates are needed, the considerations of test-fill layout should include the feasibility of locating those plates between and about test lanes in a manner which would allow sufficiently accurate determination of average foundation settlement and avoid the obstructions of plate risers in the placement and rolling of the fill. Guidance concerning use of settlement plates (see Figure 10-1) is provided in EM 1110-2-1908. A thoroughly compacted rock pad (or leveling course), 61 to 91 cm (2 to 3 ft) thick, should be placed on the foundation (whether soil or rock) prior to placing the first test lift in order to ensure that all foundation depressions and undulations are filled and a level surface is obtained. Material for the pad can be either the same rock to be used in the fill or waste rock obtained from the test quarry prior to exposing that considered to be representative of the rock to be placed in the project embankment. Placement of the pad should be in at least two lifts with rolling applied until negligible settlements are observed from level readings made on its surface.

10-2. Placement of Hard to Medium Rock

In the infancy of the transition from dumped to compacted rockfill beginning in the mid-1950's, several different methods were used to dump and spread the rock. In addition, different ideas relative to the maximum rock size which should be allowed compared with lift thickness were also evident (Sherard and Cooke 1987). In the last 15 years, the considerable experience gained in construction and performance of compacted rockfill dams has resulted in general agreement on these practices for hard

to medium rock (Sherard and Cooke 1987) as discussed below.

a. The preferred method. The preferred method for rockfill placement is to dump on the surface of the layer being placed and then to spread the layer to the desired thickness with a crawler tractor by pushing the material over the advancing face of the lift as shown in Figures 10-2 and 10-3. This procedure creates significant segregation with the larger rocks in the bottom of the lift and the smaller rock and fines in the upper part. The main advantage of this technique derives from the relatively smooth upper surface resulting from pushing the dumped rock a short distance on top of each layer being placed such that depressions and voids between larger rocks become progressively filled with small rocks and fines. This approach also facilitates maintaining the desired lift thickness because the dozer operator is always advancing the lift ahead upon the smooth surface at its proper elevation. The smooth layer also reduces tire wear, allows higher truck speeds, and provides a better surface upon which to operate the vibrating roller.

b. Contrast with past practice. Earlier rockfill placement practice attempted to avoid segregation of the rock and/or generation of fines on the lift surface to form as homogeneous a compacted mass as practicable. The procedure was to dump the truck loads of rock in piles spaced upon the surface of the previously compacted lift and then to spread the piles to form the desired lift thickness. A very irregular fresh fill surface is created which makes equipment travel difficult, rapidly wears the rubber tires, and subjects vibratory rollers to damage because they do not withstand continuous operation on irregular surfaces where the drum is pounding on a few high points of hard rock. This method of rockfill placement is now considered obsolete by most specialists (Sherard and Cooke 1987), but is still occasionally proposed.

c. Stratified rockfill is preferred (Sherard and Cooke 1987). Past practitioners viewed the generation of stratified rockfill in the placement and compaction operations to yield undesirable properties with respect to permeability and compressibility. Considerable experience with the performance of rockfill dams, whether earth-core or concrete-faced, has shown that there are no technical disadvantages to the preferred method of placement in segregated layers. Sound rock derives its typically adequate shear strength from a combination of the density of the upper-lift zone of finer particles and the larger particle wedging and interlocking in the lower-lift zone rather than strictly from density. The stratification also assures that
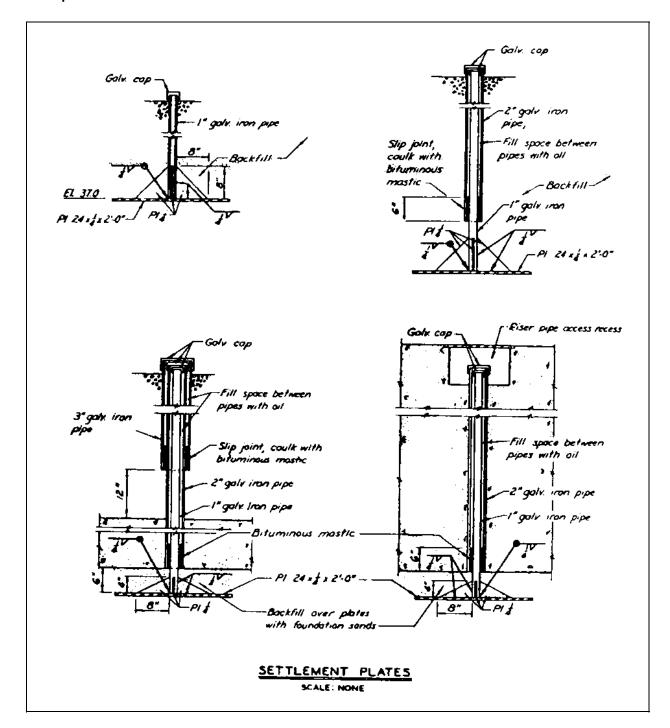
SETTLEMENT PLATES
SCALE: NONE

Figure 10-1. Typical settlement plates (from EM 1110-2-1908, Part 2)

any flow through the embankment will move much more easily in the horizontal direction than in the vertical which offers downstream slope stability advantages during construction for a concrete-faced dam if an upstream pool is impounded during construction or if there is an overtopping allowance during construction. Even for rockfill containing considerable fines, the stratified structure results in a greater average permeability compared with fill placed to a more homogeneous character.

d. Lift thickness. Lift thicknesses employed in more recent times for medium to hard rock have averaged about 1 m (3.3 ft). Cooke (1990) states that the 9.1-Mg
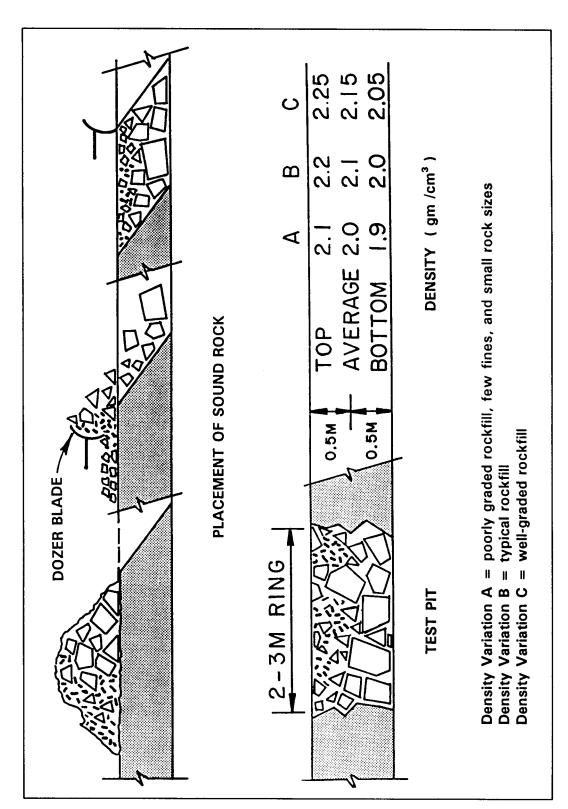
Figure 10-2. Placement of sound rockfill and resulting density variation (after Cooke 1990)

Figure 10-3. Placing a lift in the Cerrillos Dam test fill (Note the marking of the previous lift surface with plastic strips and lime)

(10-ton) vibratory roller (Figure 10-4) has generally provided excellent results for this thickness. Experience suggests that selection of lift thickness for sound rock up to about 1 m (3.3 ft.) is not a particularly critical item with respect to ability to achieve adequate compaction but can be based on quarry-run rock size brought to within the range of lift thicknesses stated above. However, the use of lift thicknesses approaching or exceeding 1 m should be on the basis of justification derived from the test fill. The literature clearly agrees that for sound rock, quarry-run material can usually be produced to be satisfactory.

Figure 10-4. A 9.1 mg (10-ton) vibratory roller at work on the Cerrillos Dam test fill

e. Maximum particle size versus lift thickness. It has been customary to limit the maximum particle size to something less than the loose-lift thickness (say, a maximum of 0.9). However, it has been clearly established that maximum particle size equal to the lift thickness is acceptable. With the preferred placement practice, the vibratory roller will seat these particles among the smaller rocks and fines. The presence of particles equal in size to the lift thickness has not been found to result in unacceptably poor compaction of intervening material, i.e., any detrimental effects on the compaction or the compressibility of the fill.

f. Grading. Sound rock is highly segregated in each lift such that grading of the quarry-run rock is not important. Cooke (1990) points out that for a given roller, well-graded quarry-run sound rock will give the highest density and modulus (lowest compressibility), but all quarry-run rock, even when poorly graded, has been satisfactory with respect to embankment performance. He further states that if the rock is hard, a satisfactory general specification is "quarry-run rock - the maximum size shall be that which can be incorporated in the layer and provides a relatively smooth surface for compaction, not more than 50 percent shall pass a 2.5-cm (1-in.) sieve, and not more than 6 percent shall be clay-sized fines." Natural gravels with sound particles do not conform to the typical definition of rockfill but may be considered for use in the shell of a dam. Loose lift thicknesses for gravels have ranged between 0.3 and 0.9 m (1 to 3 ft.) depending on particle size and percentage of minus U.S. Standard No. 200 sieve sizes (Cooke 1984).

10-3. Placement of Soft Rock

Dumped rockfill, which is still used in downstream portion of sloping-earth-core dams or in the shells of central earth-core dams, requires sound rock meeting concrete aggregate specifications. However, very low compressive strength rock such as possible in siltstones, sandstones, shists, argillite, and other potentially weak rocks may also be used as compacted rockfill. This is one of the cost advantages gained from compacted rockfill as compared with dumped rockfill in that weak rock formerly wasted from quarries for dumped rockfill dams became acceptable materials for even very high compacted embankments.

a. The preferred method. Soft (weak) rock which arrives at the test fill containing appreciable fines or which breaks down significantly in the placement

operations derives its shear strength from density so that it is generally dumped and spread by crawler tractor directly on the preceding lift to minimize segregation and yield a more compact mass. An exception would apply in cases where the breakage under the crawler tractor alters the fill from proper classification as rockfill into a soil material and alternative methods offer the possibility of retaining satisfactory rockfill traits. This statement assumes that the determination has been made that the marginal material placed in a manner retaining rockfill traits will not deteriorate into a soil material under embankment stresses or environmental factors.

b. Lift thickness. Because fill composed of soft, weaker rocks and appreciable fines derives its satisfactory properties from density, test fill results are likely to show that thinner lifts (compared with hard, durable rock) on the order of 0.46 to 0.6 m (18 in. to 2 ft) are required along with an increase in the number of passes of the roller from, say, usually 4 for hard rock to 6 or 8 for the softer, weaker rock. Some breakdown may be desirable to achieve the desired strength for these materials. The compacted mass should not exhibit any voids among larger particles, i.e., the larger particles should be consistently surrounded by finer material which has clearly been densified by the compactive effort between and among the larger particles. The use of water (to be discussed below) in the compaction of soft rock may result in the test-fill finding that somewhat thicker lifts can be used.

c. Maximum particle size. Maximum rock size may be equal to the lift thickness but these sizes will typically break down during placement and compaction.

d. Grading. Grading of weak rockfill materials is of no consequence since the procedures, i.e., lift thickness, compaction, and use of water (to be discussed below) are adopted to produce some breakdown and high density.

10-4. Rockfill Versus Soil

Weak rock introduces the possibility that it will break down during placement and compaction such that desirable shear strength and compressibility cannot be achieved unless it is compacted with water content and density control using rollers typically employed for soils. In some cases, the determination that materials will degrade to such an extent (whether by quarrying, hauling, placement, and compaction or as the result of environmental factors such as wetting or air exposure) can be made on the basis of previous experience, tests on core samples, or experience gained in the test quarry rather than resulting

from test fill observations. Otherwise, a test fill may be the only way to make the determination. It is sometimes possible to maintain the rockfill character of the material through avoidance of excessive breakage in the fill operations by processing the quarry-run material to remove fines and smaller sizes, by using rubber-tired equipment for the spreading operation, by adjusting roller passes, weight, or vibration settings, or by a combination of these. If excessive fines exist in the materials or if they are generated during compaction, the vibratory roller may be less effective in compaction compared with the 45.4-Mg (50-ton) or 68.0 Mg (75-ton) pneumatic roller. In general, materials which retain the properties to be properly called rockfill can be brought to satisfactory density using a vibratory roller. Superior compaction by a pneumatic roller would probably indicate that the material is more properly classified as a soil and should be treated as such in design, construction, and construction control. Perhaps the key concept distinguishing rockfill from that of soil is that the rock particles are in contact within the compacted mass as opposed to "floating" in a "matrix" or "binder" of soil-sized material (i.e., sands to silts or clays).

10-5. Use of Water

The use of water in compaction of rockfill (Figure 10-5) is beneficial no matter what the rock quality, but becomes especially important for types of rocks which exhibit strength loss upon wetting (usually indicated by low compressive strength) or whenever there is an appreciable presence of fines. The use of water in the compaction of weak rock, whether or not some breakdown is a desirable end, has been general practice. Indeed, one of the additional indicators as to whether or not the soft-rock material retains rockfill properties, is whether or not the rockfill is strong enough to support hauling equipment and the vibratory roller when wetted to saturation. If the equipment becomes immobile, the material ruts more than several inches under the tires of the trucks, or the added water stands upon the surface, the material has soil strength, not rockfill strength. This observational approach is valid also for hard and medium rock if excessive fines are present. The application of water has been on the order of 15 to 20 percent of the volume of the material. The use of water may represent a serious environmental factor if drainage from the fill creates turbidity pollution of the stream or river. Where water use is restricted for environmental or economic reasons, Cooke (1984) cites alternative practice of placement of weak rock in thinner lifts of 0.6 m (2 ft) or less along with an increase in the number of passes of the vibratory roller. For any weak rock which exhibits a significant loss of
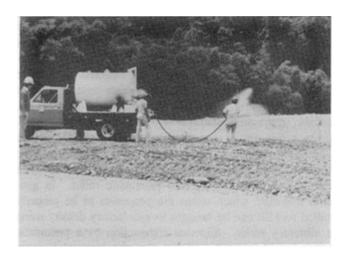
Figure 10-5. Applying water prior to lift compaction on one of the Cerrillos Dam test fills containing appreciable fines (Note: The operation shown above was an expedient method for the test fill at the Puerto Rico damsite. More typically, water is applied from a pressurized tanker truck with a rear spray bar)

strength for saturated specimens, a saturated test fill should be conducted to establish placement and compaction specifications.

10-6. Compaction and Compaction Equipment

After the rock has been placed in the desired lift thickness, the compaction operation is begun. Where surface settlement readings are to be used to assess densification (the typical practice), it is advantageous to smooth the surface of the lift for marking of the settlement grid by making one complete coverage with the vibratory roller with the vibrating unit off. The procedure for settlement readings is addressed in Chapter 11. As has been previously stated, it is important that the compaction operation be accomplished in a manner to simulate anticipated project procedures, except for interruptions required to make measurements and observations. Each pass of the roller, whether vibratory or rubber-tired pneumatic, should overlap the previous pass by about 0.3 m (1 ft). Specifics regarding the vibratory and pneumatic roller are discussed in the following paragraphs.

a. Vibratory roller. Vibratory rollers (Figure 10-4 and Appendix B) have evolved considerably since their inception in the mid-1950's. It is important for test-fill designers and field personnel to become familiar with current manufacturers' literature and recommendations for operational speed versus frequency settings to obtain the most efficient compaction. Appendix B contains recent information obtained by Los Angeles District pertaining to

these parameters and the specifications they instituted for Seven Oaks Dam. The amplitude of the roller is the distance the drum lifts off the ground in its vertical vibration and the frequency is the number of times per minute it lifts off the ground (i.e., number of impacts) expressed as vibrations per minute or VPM. Numerous studies on distance between successive impact points and centrifugal force have been conducted over the last 20 years which have resulted in the establishment of 6 to 8 impacts of the drum per lineal foot of travel as a minimum for optimal performance. Appendix B presents a table of VPM versus impacts per lineal foot for different speeds of operation of the roller. A modern roller can operate at a frequency of 1500 to 1800 VPM delivering forces in excess of 4.1 Mg (9,000 lb) per 0.3 m (1 ft.) of drum width. Increased frequency translates to increased speeds of operation which represents construction cost savings. With increased frequency, a greater force is applied and more impacts per lineal foot of rolling can take place. As part of the test fill evaluation objectives, these variables can be adjusted to provide the optimum rolling procedures for the given material and offer some latitude to alter the degree of breakdown if it is a problem.

b. Pneumatic roller. Bertram (1973) suggests specifications for a 45.4-Mg (50-ton) rubber-tired roller as follows: "Pneumatic rollers shall have a minimum of four wheels equipped with pneumatic tires. The tires shall be of such size and ply as can be maintained at tire pressures between 552 kPa and 690 kPa (80 and 100 psi) for a 11.3-Mg (25,000-lb) wheel load during rolling operations. The roller wheels shall be located abreast and shall be designed so that each wheel will carry approximately equal load in traversing uneven ground. The spacing of the wheels shall be such that the distance between the nearest edges of adjacent tires will not be greater than 50 percent of the width of a single tire at the operating pressure for a 11.3-Mg (25,000-lb) wheel load. The equipment shall be subject to the approval of the contracting officer." Pneumatic rollers should be towed or operated at speeds less than 8 kmph (5 mph). Heavier pneumatic rollers are now available and should be considered as applicable but documentation of their use on rock test fills has not been discovered. For most test fills, the optimum performance of either a pneumatic or vibratory roller has been achieved in 8 passes or less. Typically, the compaction program for any given lift thickness has been to schedule a maximum of 8 passes (lift coverages) with interruptions between each two passes for measurements and observations. After compaction of a given lift has been completed and all tests and measurements have been made, the surface of the completed lift may be covered with a marker material such as lime or a heavy

plastic membrane (0.02 cm or 8 mil maximum thickness) as shown in Figure 10-3 to facilitate identification of individual lifts within an inspection trench or pit after the entire test fill is complete. Inspection trenches or pits will be discussed in Chapter 11.

Chapter 11
Measurements and Observations

11-1. General

Both measurements and visual observations are of importance since the overall conclusions reached from the results of a test fill program are as much qualitative as quantitative. The importance of good diary keeping and photographic records cannot be overemphasized, especially in view of the fact that design personnel who are to use the information usually cannot be present at the site at all times. Like the layout and design of test fills, the measurements and observations made are highly dependent on the primary objectives of the fill program. Advance planning and scheduling of tests are an integral part of the overall design. In this respect, flexibility is also important, since only rarely can the test program be fully laid out beforehand and carried out with no deviations. Provisions should be made for supplemental tests and for relocation, if necessary, of the test sites. Personnel who are to conduct the tests should be made familiar with the program and procedures. Personnel should also be made aware of what is expected of them as far as visual observations are concerned. It is highly desirable for a representative of the design group to be present at all times.

11-2. Densification

The densification of rockfill may be judged by: measuring the settlement resulting from compaction, performing in situ density tests, detailed observations within inspection trenches, and a combination of the preceding items. Because of the difficulty and expense of conducting enough tests to ensure representative results and because results of in situ density tests are sometimes questionable (especially for large rock), such tests should not be relied upon as the sole means of judging the effectiveness of the compaction process. Settlement determination by methods subsequently described should be used for this purpose in conjunction with visual observations in inspection trenches and with in situ density tests when available. In situ density tests are useful in that they provide quantitative values and allow comparison with the densities of other lift thicknesses or materials to be made but they are time-consuming and expensive to conduct.

a. Settlement. Settlement of the fill surface is measured by taking level readings at many points on the test section in a grid pattern. A 1.5- by 1.5-m (5- by 5-ft) square grid has often been used for this purpose. A

1.2- by 1.8-m (4- by 6-ft) or 1.5- by 2.1-m (5- by 7-ft) grid has also been used depending on the shape of the test fill area. Any gird pattern is acceptable as long as enough points are provided to obtain a good representative assessment of the overall settlement of the lift surface. There should be no less than 3 points on any one line of the grid and the edges of the grid should be no closer than 3 m (10 ft) from any outside edge of the test section to avoid settlement readings in an area where the rolling of the fill may have caused bulging. The areas to be avoided for settlement readings also include those next to the access ramps where lateral movement may also occur against the random, more compressible material often placed in those areas. Examples of settlement grid layouts are shown in Figure 11-1.

(1) Prior to establishing the grid points on the uncompacted lift surface, a leveling pass should be made by the vibratory roller with the vibratory unit off. This will provide a smoother surface upon which to establish and mark the grid points and to confirm the loose-lift thickness. This leveling pass with the vibratory roller can also be used when other types of rollers are to be assessed.

(2) There are several methods to establishing the grid. In most cases, wires or strings have been pulled from perimeter stakes set at the desired spacing. Another satisfactory method has utilized a light-weight template consisting of a metal frame strung with wire or twine. In any event, after the points are located, they should be well marked on the fill surface with contrasting paint to facilitate identification for subsequent level readings.

(3) Since the reading at a point must represent the area surrounding it (for points on 1.5-m (5-ft) centers, for instance, each point represents a 1.5- by 1.5-m (5- by 5-ft) area), it is important that the level rod be placed where it is indeed representative of this area and not influenced by local irregularities at the point. This is an expectable problem on a rockfill surface which can be ameliorated by use of the device shown in Figure 11-2. The simple device consists of a 0.3-m (1-ft) square metal plate with a raised button in its center upon which the level rod is seated for readings. A handle made from a steel rod is attached to the plate to help in firmly seating it and transporting it from point to point. The leveling instrument should be located carefully with regard to equipment trafficking so as to avoid its disturbance throughout placement and rolling operations. Bench marks should be established in secure places well away from the fill area. These are well known good survey

Settlement reading locations, Gathright Dam test fill

Grid for settlement readings, New Melones Dam test fill
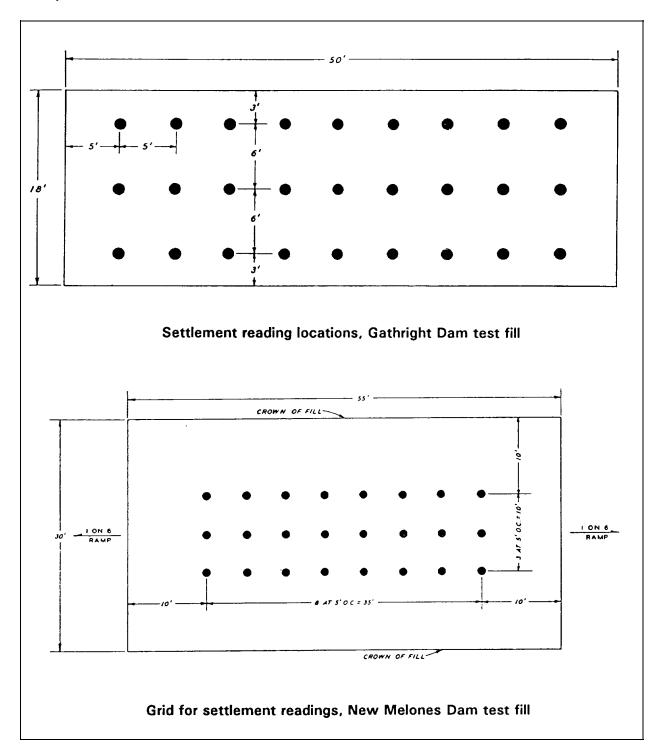
Figure 11-1. Example settlement grid layouts

practices but reports on past test fills have attributed erratic settlement results to disturbance of the level instrument in more than a few cases.

(4) The settlement of a particular lift is obtained by averaging the settlements measured at all points in the grid usually expressing it as a percentage of loose-lift

Figure 11-2. Apparatus for taking level readings

thickness. If a number of passes of the roller is a variable being evaluated, readings of the grid points are usually made after every two passes or coverages of the roller. One pass of a pneumatic or vibratory roller is equivalent to one coverage. Those points that indicate heave rather than settlement may be eliminated from the averaging process if none of the surrounding points indicate heave or if it is noted (by careful observation and documentation by the rodman) that a local condition at the point is not representative of the surrounding area. If settlement plates have been installed beneath the fill on a compressible foundation, they should be read at the same frequency as the surface grid points, and any settlement they indicate averaged and subtracted from the lift surface settlement.

b. In situ density tests. The typical means of performing an in situ density test in rockfill is by excavating a test pit in the fill, salvaging and weighing the removed material, and determining the volume of the pit by the water-volume method. The volume of material excavated is necessarily large in order to minimize the effects of the larger particles on the results. The test pit may be either square or round in plan shape and a rigid template of wood or metal is anchored on the fill surface as a guide in excavating the hole. The template size and, therefore, the volume of the test pit varies on the basis of the maximum particle size in the compacted material. The most popular template seen in the rockfill literature has been round and metal. For that reason, in situ density tests in rockfill are often referred to as "ring densities." Since rockfill is segregated or stratified, the test pit should be excavated through the entire lift to obtain an average density. The use of lift marker material such as lime or

plastic sheeting previously mentioned in paragraph 10-5 proves its worth in clearly indicating the lower lift boundary. Detailed guidance in conducting the test is now provided as ASTM Designation D 5030 - 89 (see ASTM 1994a). Weighing of the total sample removed form the density test typically requires high-capacity scales and may even involve weighing of individual larger particles. Rockfill materials are typically not placed under density and/or water-content specifications as are soils but are placed under method-type specifications with no water content control. The in situ density test is not used as a construction control test in project construction simply because of its time-consuming nature, the fact that rockfill density numbers have little specific meaning in design and the method-type specification coupled with good observation is relied upon for compaction quality. This is precisely why a test fill is so important, i.e., to aid in developing the proper method specification. The value of the test is primarily as a basis of comparison of compaction procedures in a test fill in establishing the method specification, i.e., lift thickness and number of passes by a specified roller, and to obtain as-built data during construction. Because of the heterogeneity of rockfill, in situ tests taken in the same lift at different locations can be expected to yield different results although more than one test per lift is rarely ever performed to reveal that expectable occurrence. In some cases, in situ tests have been employed in project construction, not only to obtain as-built data, but also when changes in materials or compaction results are suspected. U.S. Committee on Large Dams (1988) presents the experiences of several agencies, including the Corps of Engineers, with respect to in situ density testing.

c. Laboratory maximum density test. It has not been uncommon practice to compare the results of in situ density tests with the results of some version of vibrated laboratory maximum density. Both the Corps of Engineers (EM 1110-2-1906) and ASTM Designation D 4253-93 (ASTM 1994c) provide standard procedures for cohesionless materials with a maximum particle size of 7.6 cm (3 in.). EM 1110-2-1906 allows the scalping of up to 10 percent by weight of particles larger than 7.6 cm (3 in.) but ASTM requires that 100 percent of the material is smaller than 7.6 cm (3 in.). The literature such as U.S. Committee on Large Dams (1988) describe various non-standard large scale vibrated tests in large molds. The U.S. Army Engineer District, Los Angeles (1992) performed saturated tests in a 68.5-cm (27-in.) diameter mold on minus 15.2-cm (6-in.) scalped fractions of Seven Oaks Dam test fill gradations. The U.S. Army Engineer Waterways Experiment Station conducted an unpublished and limited test program on minus 15.2-cm (6-in.) rock

employing standard tests in the 27.9-cm (11-in.) diameter mold on minus 7.6-cm (3-in.) scalped fractions and full-scale gradation tests in a 1.83-m (6-ft) diameter mold vibrated with an MTS® system actuator. That test series showed that the results of the standard test could be corrected for the oversize (plus 7.6-cm (3-in.) fraction) using the following equation of EM 1110-2-1911, Appendix B (Ziegler 1948):

$$\gamma_f = \frac{f\gamma_t\gamma_w G_m}{\gamma_w G_m - c\gamma_t} \qquad (11\text{-}1)$$

or, solving for γ_t:

$$\gamma_t = \frac{\gamma_f\gamma_w G_m}{f\gamma_w G_m + c\gamma_f} \qquad (11\text{-}2)$$

where, for this particular case:

γ_f = maximum dry density of the minus 7.6-cm (3-in.) fraction obtained from the standard EM 1110-2-1906, Appendix XII, vibratory table test, metric ton/m^3 or lb/ft^3

f = percentage by weight of the total sample represented by the finer fraction, i.e., the minus 7.6-cm (3-in.) fraction

γ_t = calculated maximum dry density of the total material, metric ton/m^3 or lb/ft^3

γ_w = unit weight of water, 1 metric ton/m^3 or 62.4 lb/ft^3

G_m = bulk specific gravity of the oversized fraction (plus 7.6-cm or plus 3-in.), see EM 1110-2-1906, Appendix IV

c = percentage by weight of the total material represented by the coarser fraction, i.e., the plus 7.6-cm (3-in.) fraction

11-3. Gradation Tests

Gradation tests are used to determine the amount of breakage the rock has suffered during placement and compaction. This is accomplished by running gradation tests on samples representative of the material as

delivered to the fill ("before" gradations) and on samples taken from the compacted fill ("after" gradations). Differences in the two resulting curves indicate the degree of breakdown of the material. After-compaction samples are usually obtained from material excavated from the fill density test pit or from the side walls of inspection trenches or test pits. Again, it is important that the entire thickness of a lift be sampled. At the present time there is no standard procedure for obtaining the gradation of rockfill materials. A gradation sample must also be large in order to obtain a representative gradation curve with reasonable accuracy in the results. One approach to the concept of accuracy is to consider a test sample of such size that the addition or loss of the largest particle will not alter the "percent finer by weight" by more than an acceptable number of percentage points (i.e., shift the gradation curve coarser or finer). For instance, for a test sample to approach 1 percent accuracy, assume that the largest particle weighs 68 kg (150 lb). For this rock to correspond to less than 1 percent of the total sample weight, the total gradation sample size would have to be 6.8 metric tons (15,000 lb). In a similar manner, for 2 percent accuracy, the sample would have to weigh 3.4 metric tons (7,500 lb). From a practical point of view, a 1 percent accuracy is probably an extreme requirement and a 2 percent accuracy is a reasonable minimum criteria. The difficulty for rock materials lies in handling the heavy sample, obtaining its total weight, dividing it into fractions for gradation by different procedures, and then mathematically recombining the results on the fractions into a single gradation curve. A large, clean area (preferably a concrete slab) is needed to spread out the larger particle fraction. Determining the percentage by weight of total sample which the largest particles in various size ranges represent typically requires hand measurement in some manner of the size of larger particles and the summing of their weights for selected size ranges. Procedures for rocks larger than 12.7 cm (5 in.) now provided in ASTM Designation: D 5519-93 (ASTM 1994d) make be considered applicable for rock fill materials though specifically they are directed at riprap. For the total material fraction smaller than that treated by ASTM Designation: D 5519-93 (ASTM 1994d), gradation test procedures for aggregates such as Designation: C 136 (ASTM 1994b) and for soils such as Designation: D 422 (ASTM 1994c) or EM 1110-2-1906, Appendix V are available. The U.S. Committee on Large Dams (1988) describes the past large-scale gradation practices of several agencies which may also still be considered applicable. The procedures used in the construction of

Carter's, Cerrilllos, and Seven Oaks Dams were described previously in paragraph 6-5.

11-4. Percolation Tests

Rockfill in a dam shell is assumed to be a free-draining material in design. In cases where the material breaks down such that it exhibits poor to practically impervious drainage characteristics, the zoning of the embankment should ordinarily specifically provide for the use of such materials. It may be one of the objectives of the test fill to determine if such embankment design features must be incorporated to efficiently use available materials. For hard to medium rock, there is rarely a need to perform percolation (infiltration) tests to verify the free-draining characteristics. Assessments of the drainage characteristics of rockfill are very crude and are properly termed "percolation" or "infiltration" tests as opposed to "permeability" tests. Field methods applied in the test fill can only yield a very rough estimate of permeability because, among other factors, the material is unsaturated and the area of flow discharge is not known. Furthermore, the variability of the rockfill itself and that of permeability determinations (even under the best of laboratory conditions) would likely result in different values at different locations in the same lift of orders of magnitude. It has become customary to describe soil-rock materials with permeabilities less than 0.3 m/year (1 ft/year) as impervious, those with permeabilities between 0.3 and 30 m/year (1 and 100 ft/year) as semipervious, and those with permeabilities greater than 30 m/year (100 ft/year) as pervious (U.S. Department of the Interior, Bureau of Reclamation 1985). These ranges were derived from Terzaghi and Peck (1948) who described them as representing good drainage characteristics if permeability is greater than 10^{-4} cm/sec (103 ft/year), poor drainage characteristics if permeability is between 1×10^{-6} cm/sec and 1×10^{-4} cm/sec (1 and 103 ft/year), and practically impervious if permeability is less than 1×10^{-6} cm/sec (1 ft/year). The following paragraphs describe several versions of a percolation test given in the order of decreasing applicability for most test fill situations.

a. Open pit method. This method has been by far the most commonly used. Percolation tests are usually performed in either the in situ density test hole after removal of the plastic liner employed to obtain the water-volume or in a separate pit at least 1-m (3-ft) square and at least one lift thickness deep. If the pit will not retain a sufficient volume of water to measure the rate of fall of the water surface, the compacted material is obviously free-draining. If the pit can be filled such that a rate of fall of the water surface can be measured, the percolation

rate (the distance the ponded water surface falls in the pit over a measured time) in centimeters/second, meters/day or ft/day may be taken directly as a very rough indication of the permeability. This simple approach will overestimate the permeability and further guidance concerning its use given in paragraph 11-4c below should be considered. However, if this crude method indicates that the drainage characteristics are less than free-draining, it is a reliable assumption that they are. Justo (1991) provides a more theoretical method to estimate a permeability from the rate at which the water surface in a test pit falls, i.e., a falling-head test. The permeability is calculated with reference to Figure 11-3 and using the following equation. An example calculation is given below in paragraph 11-4c.

$$ k = - \frac{\Delta h}{\Delta t \ (1 - n \ \Delta S)} - \frac{h_0 \ n \ \Delta S}{\Delta t \ (1 - n \ \Delta S)^2} $$
$$ ln \frac{h \ n \ \Delta S - \Delta h}{h_0 \ n \ \Delta S} \tag{11-3} $$

where, for any consistent set of units:

k = coefficient of permeability

h_0 = the initial depth of water in the pit at time $t = 0$ sec

h = depth of water in the pit after a time Δt sec

$\Delta h = h_0 - h$ = change in depth of water in the pit during time interval Δt sec. **Δh takes a negative sign in the above equation because in its derivation, as the head decreases during the test, the volume of water in the material from the bottom of the pit to the wetting front is increasing.**

n = the porosity of the fill beneath the pit, dimensionless

ΔS = change in degree of saturation (expressed as a decimal value) of the material below the pit from its initial value before filling of the pit to its wetted value during the test.

This approach assumes that the pit is filled instantaneously which is not the practical case and the change in degree of saturation must be estimated. Furthermore, flow is assumed to take place only in the vertical
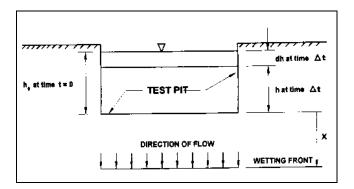
Figure 11-3. Open-pit percolation test after Justo (1991)

direction such that at any distance x below the pit (see Figure 11-3), the area of discharge is not assumed to have increased. This assumption represents a sever simplification of the flow pattern since flow will occur through the side walls of the pit below the water level and will also spread out laterally beneath the pit with distance x in downward movement of the wetting front. Since increase in discharge area with distance below the pit is not considered, the calculated value of permeability will be larger than the actual value to some unknown degree assuming that all other parameters entered into the equation are correct. The initial degree of saturation of the fill and its porosity can be calculated according to EM 1110-2-1906, Appendix II, from a knowledge of the initial water content of the fill, the specific gravity of the material, and the compacted density of the fill. This information can be obtained as part of the conduct of a fill-density test. If the fill contains more than about 10 percent by weight of particles passing the U.S. Standard No. 4 sieve, the specific gravity of the material should be calculated using the equation given in EM 1110-2-1906, Appendix IV, paragraph 3e, for materials consisting of both plus and minus No. 4 sieve fractions. The wetted degree of saturation of the fill beneath the test pit at the time Δt during conduct of the percolation test must be estimated. The wetted degree of saturation may vary between about 75 and 95 percent. A reasonable value to assume for the computation of permeability is 85 to 90 percent.

b. Standpipe methods. The standpipe methods to be described below consist of using a cased hole (i.e., an implanted metal or plastic pipe in the case of the rock test fill) to perform either a constant-head or falling-head permeability test. The falling-head standpipe test is probably the most technically sound method among the four methods presented because it does consider the increase in area of flow (to be discussed below) after a fashion proven in model studies. The standpipe tests are applicable in materials exhibiting many times the permeability

identified previously as indicating good drainage characteristics. However, they should not be used unless the fill contains appreciable fines and the considerable time and costs are deemed to be justified. The fill should probably exhibit a minus No. 4 sieve fraction exceeding 30 percent by weight or a minus No. 200 sieve fraction of 10 percent or more by weight in order that it not be so permeable as to outstrip the practicality of a standpipe test. If the material is gap-graded, the question of its permeability outstripping the practicality of the standpipe test is complex but if the finer fraction does not fill the voids between the larger particles, the permeability is likely to be very high. There are no firm guidelines concerning the diameter of the pipe but ideally it should not be less than twice the diameter of the maximum particle size of the material after compaction. If this criterion dictates a very large (in practical terms) pipe diameter, say, exceeding about 61 cm (2 ft), a ratio of diameter to maximum particle size of less than two may be used but with care taken to avoid a single particle immediately below the tip of the pipe which would constrict flow. The objective is to test the mass of the compacted rock to the maximum practical extent. The pipe should be inserted into the fill to a sufficient depth such that it is stable in position. In general, the pipe must be placed in an excavation kept to minimal working dimensions and then backfilled about its exterior with the excavated material with some attempt to maintain its approximate gradation as it was in situ. Material above the level of the tip of the pipe has some impact on results because the flow of water has been shown by model tests to develop a wetting front in the unsaturated material which is approximately spherical about the tip of the pipe.

(1) Schmid (1967) provides a method for estimating permeability from the results of a falling-head test performed above the ground water table (in unsaturated material) in a standpipe. Because the volume of the standpipe is small compared to the ability of the material to consume the flow, the depth of pipe embedment need only be that sufficient to stabilize it. Figure 11-4 shows the configuration of the test and the spherical wetting front (documented by model tests) which develops as the water flows from the pipe. The equation Schmid (1967) derived is given below. An example calculation is given in paragraph 11-4c.

$$k = \frac{r_0}{4} \frac{\ln \frac{h_1}{h_2}}{t_2[\frac{3 (h_1 - h_2)}{4 n \Delta S r_0} + 1]^{\frac{1}{3}} - t_1}$$ (11-4)
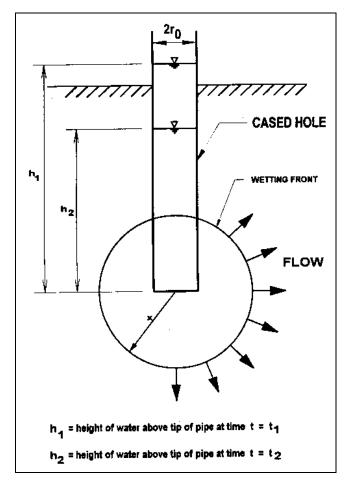
Figure 11-4. Falling-head standpipe percolation test after Schmid (1967)

where for any consistent set of units:

k = coefficient of permeability

r_0 = inside radius of the standpipe

h_1 = height of the water above the tip of the stand-pipe at time $t = t_1$

h_2 = height of the water above the tip of the stand-pipe at time $t = t_2$

ΔS = change in degree of saturation of the rockfill from its initial value to its wetted value expressed as a decimal value.

n = the porosity of the rockfill

The wetted degree of saturation can again be assumed to be 85 to 90 percent as previously stated in paragraph 11-4a.

(2) The U.S. Bureau of Reclamation in its *Earth Manual* (USBR 1985) provides an open-end field permeability measurement method as Designation E-18. The method consists of measuring the flow rate of water required to maintain a constant head in the pipe under a constant rate of flow. Gravity flow in an open standpipe or pressurized (pumped) flow in a sealed standpipe may be used. For this test, the pipe must be inserted into the fill to a depth such that the head of water applied as measured from the tip of the pipe is less than the depth of embedment. Otherwise, the flowing water may rise about the pipe to exit upon the surface of the fill which will invalidate the already approximate test method. This constant-head test is complicated for use in rock test fills by the need to measure and maintain the flow rate, and by the necessity to maintain either a relatively constant water level in the pipe (gravity flow) or a relatively constant pressure (pressurized flow). In tests using gravity flow, a constant water level in the standpipe is rarely maintained in unsaturated materials and a surging of the level within less than 15 cm (6 in.) at a constant rate of flow for about 5 minutes is considered satisfactory. In the pressurized test, this acceptable head variation corresponds to a pressure variation of only about 21 kPa (3 psi). The equation used to calculate permeability was derived by the USBR from electrical analogy studies and is given below (see Figure 11-5). An example calculation is given in paragraph 11-4c.

$$k = \frac{Q_d}{5.5 \ r_0 \ H_w} \tag{11-5}$$

where, for any consistent set of units:

k = coefficient of permeability

Q_d = flow rate

H_w = applied head

r_0 = inside radius of the standpipe

c. A comparison of the methods. Justo (1991) compares results of open-pit and standpipe percolation
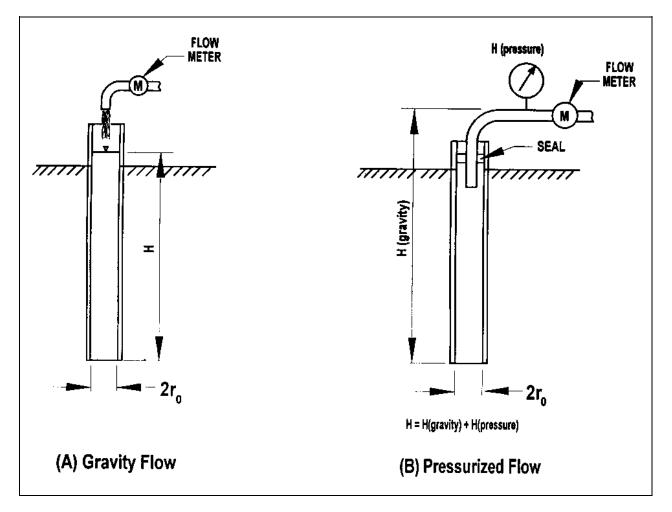
Figure 11-5. USBR gravity flow and pressurized flow standpipe percolation tests

tests performed in the test fill for Martin Gonzalo Dam, Cordoba, Spain. The open-pit method was conducted in a pit excavated to dimensions of 1m square by 0.25 m deep (3.3-ft square by 9.8 in. deep). Both the USBR constant-head, constant-flow test and the falling-head test were performed in standpipes. The permeabilities according to the equations (11-3) through (11-5) presented above in paragraphs 11-4a and 11-4b are calculated below.

(1) The open pit was filled to an initial depth of 25 cm (9.8 in.), i.e., to the surface of the test fill. Parameters pertinent for entry into Equation (11-3) of paragraph 11-4a were as follows:

dry density of the fill = 2.14 Mg/m^3

porosity of the fill = n = 0.20

initial degree of saturation S_i = 10 percent = 0.10

wetted degree of saturation S_w = 90 percent = 0.90

$$\Delta S = (S_w - S_i) = (0.90 - 0.10)$$
$$= 0.80$$

$$1 - n\Delta S = 1.0 - (0.20)(0.80) = 0.84$$

$$h_0 = 25 \text{ cm}$$

Δh during conduct of the test was 11 cm (-4.33 in.) over a time period Δt = 180 sec. It is important to apply a minus sign to Δh in the equation (11-3). Because Δh observed was 11 cm (4.33 in.), h at time Δt = 180 sec = $h_0 - \Delta h$ = 25 cm - 11 cm = 14 cm (5.51 in.)

Substituting into Equation (11-3):

$$k = - \frac{-11}{180 \times 0.84} - \frac{25 \times 0.16}{180 \times (0.84)^2}$$

$$ln \frac{(14 \times 0.16) - (-11)}{25 \times 0.16}$$

(11-6)

or:

$$k = 0.0728 - 0.0377 = 0.0351$$

$$= 3.51 \times 10^{-2} \text{ cm/sec} = 1.1 \times 10^4 \text{ m/yr}$$

The material is free-draining according to the criteria discussed in the first part of this section because the measured permeability is a great deal in excess of 30 m/year (103 ft/yr).

(2) The parameters from the Martin Gonzalo Dam tests pertinent for substitution into Schmid's Equation (11-4) of paragraph 11-4b(1) were as follows:

As before

$$n\Delta S = 0.16$$

radius of the standpipe $r_0 = 10.25$ cm

$$h_1 = 40 \text{ cm}$$

$$h_2 = 20 \text{ cm}$$

$$t_1 = 0$$

$$t_2 = 192 \text{ sec}$$

substituting into Equation (11-4):

$$k = \frac{10.25}{4} \frac{ln 2}{192[\frac{3 \times 20}{4 \times 0.16 \times 20.5} 1]^{1/3}}$$

(11-7)

$$k = 5.2 \times 10^{-3} \text{ cm/sec} = 1.64 \times 10^3 \text{ m/yr}$$

(3) With respect to the USBR standpipe method, the constant flow measured for a constant head of 30 cm was 34.38 cm^3/sec in the same standpipe with $r_0 = 10.25$ cm. Substituting into equation (11-5) yields:

$$[k = \frac{34.38}{5.5 \times 10.25 \times 30} = 2.03 \times 10^{-2} \text{ cm/sec}$$

$$= 6.40 \times 10^3 \text{ m/yr}$$

The values of permeability yielded by the three tests given above vary by about one order of magnitude (one power of 10) which is an excellent result considering the differences in the tests and the partially saturated state of the fill. The permeability of a partially saturated soil is very sensitive to the pore space filled with water or the so-called "effective porosity." Even permeability tests performed on replicate specimens (specimens carefully prepared to be as identical as practicable) of a saturated soil in a laboratory environment using different methods of test and calculation of coefficient of permeability may yield results of greater variation than seen in the above examples. It is instructive to note from the open-pit test above that if the permeability is estimated by simply the fall of the water $\Delta h = 11$ cm divided by the time lapse $\Delta t = 180$ sec, i.e.:

$$k = \frac{11}{180} = 6.11 \times 10^{-2} \text{ cm/sec}$$

$$= 1.93 \times 10^4 \text{ m/yr}$$

(11-8)

the resulting number is on the order of 2 times the value yielded by the Justo open-pit method to 10 times the value obtained from the Schmid falling-head standpipe method. Since the Justo method overestimates the permeability for reasons previously stated, it is recommended that the permeability estimated directly by fall of the water Δh divided by the time lapse Δt be reduced by a factor of 10 in judging the actual drainage characteristics of the fill.

11-5. Other Tests

It is beyond the scope of this manual to provide a treatise on the subject of rock testing. Most current laboratory tests do not have direct correlative relationships to rockfill with respect to either rock quality specifically as a fill material or its engineering properties or behavior. For high-quality, hard to medium rockfill materials, laboratory shear tests, compressibility tests, and associated stability analyses are typically not performed for the embankment itself and embankment slopes are adopted at 1 vertical on 3 horizontal since the shear strength of sound rockfill is

well established to be at least ϕ = 45 degrees. The designer must make a decision regarding rockfill which breaks down significantly as to whether it still retains rockfill strength or must be treated as a soil containing large particles, in which case embankment stability analyses may be required. Of course, in the event that there exists the possibility that embankment failure may involve the foundation, stability analyses are conducted for those cases regardless of the quality of the rockfill. The comments provided below are very general and divided into the categories of laboratory tests and field tests.

a. Laboratory tests. There are a number of index-type tests which may be performed on rock to obtain a judgment of its mechanical or environmental durability. In addition to the tests previously mentioned in Part 1 of this manual (some are ASTM Standards), there is the Los Angeles Abrasion test for large aggregate described in ASTM Designation C 535-89 (ASTM 1994b). That test is restricted to minus 7.6-cm (3-in.) particle sizes and has limited usefulness in indicating the likely breakdown of material under placement and compaction operations. Hammer and Torrey (1973) attempted to analyze available data from test fills to correlate Los Angeles Abrasion data to degradation during handling and compaction. Their attempts were not successful at that time and no subsequent correlations are known to exist based on the improved procedures of ASTM C 535-89. At least it is reasonable to assume that rock which suffers serious degradation in the test will suffer significant breakdown in rockfill operations. EM 1110-2-2302 lists several tests and criterion for suitability of stone for general construction use (see Table 11-1). Those criterion are very general and should be considered to indicate the higher quality rockfill materials which may be relatively obvious without test results in many cases and, therefore, do not help much in prediction of rockfill qualities of softer materials. Other references pertinent to rock and rockfill quality testing are Lutton, Houston, and Warriner (1981), U.S. Army Engineer Waterways Experiment Station (1993), and NATO ASI Series (1991). Donaghe and Torrey (1985) and Torrey and Donaghe (1991) treat the shear strength and compaction characteristics of earth-rock mixtures (soil materials). Lutton (1977) and Strohm, Bragg, and Ziegler (1978) address shale materials which may appear as rockfill upon excavation and placement but which degrade with wetting into a soil.

b. Other field tests. In situ tests (other than in situ density and percolation tests) may be performed in the test fill to assess the strength and compressibility of the compacted material although such tests have very rarely been used in this country. As described by Justo (1991),

compressibility has been assessed by large plate load tests and shear strength has been measured by plate load tests taken to failure and in situ passive failure shear tests (jacking a vertical plate against a vertical rockfill face).

11-6. Visual Observation

Because of the nature of a test fill, visual observation of the various construction procedures and material behavior are very important as a source of qualitative supplemental information. Some items meriting close observance are: preparation of the leveling pad before fill construction, installation procedures for any instrumentation such as settlement plates, character of the rock delivered to the fill such as consistency in gradation and condition, breakage of the rock during spreading relative to the degree of working by the crawler tractor, breakage of the rock during compaction, effects of added water (if any), smoothness of the surface after each interval of rolling, appearance of the fill during and after rainfall, and any variation in established behavior of any phase of the construction operation. All visual observations should be well documented with photographic evidence and a written record.

11-7. Inspection Trenches or Pits

It is highly desirable to expose a cross section of each test section or lane in order that general in situ characteristics of the compacted fill might be observed. This is achieved by the excavation of pits or inspection trenches. The inspection pit is excavated from the top down through a lift immediately after rolling or through all or part of the lifts after the entire test section or lane is completed. An in situ density test excavation can also serve as an inspection pit. If separate inspection pits are employed through several lifts, they should be large enough in plan area to permit the safe presence of personnel to inspect the side walls and even take samples. An inspection trench is a cut made through the entire depth and usually across the entire width of the completed test section or test fill. Excavation is normally with a front-end loader or dozer. The inspection trench is most often used, since the only advantage of an inspection pit is that it can be dug at any stage during the rolling operation, but this is not often justified. Except as a source of after-compaction gradation samples, an inspection trench is primarily for qualitative examination such as amount of rock-to-rock contact of the compacted material. Figure 11-6 shows such an inspection trench cut through the test fill for New Melones Dam. That trench was excavated approximately along the line of cross section A-A of Figure 9-5. Note

Table 11-1
Criteria for Evaluating Stone (after EM 1110-2-2302)

Test	Approximate Criterion for Suitability*
Petrography	Fresh, interlocking crystalline, with few vugs, no clay minerals, and no soluble minerals
Unit Weight	Dry unit weight 160 lb/cu ft or greater
Absorption	Less than 1 percent
Sulfate Soundness	Less than 5 percent loss
Glycol Soundness	No deterioration except minor crumbs from surface
Abrasion	Less than 20 percent loss for 500 revolutions
Freezing-Thawing	Less than 10 percent loss for 12 cycles
Wetting-Drying	No major progressive cracking
Field Visual	Distinctions based on color, massiveness, and other visual characteristics
Field Index	Distinctions based on scratch, ring, and other physical characteristics
Drop Test	No breakage or cracking
Set Aside	No breakage or cracking after one season cycle

* Marginal test results usually indicate the need for supplemental testing for definitive evaluation

from Figure 11-6 that the trench is of such a width compared with the maximum height of fill as to ensure the safety of personnel moving about within it to observe the side walls or take samples. Regardless of the size of inspection pits or trenches, personnel entering them should be screened for the proper and usually required construction safety equipment such as steel-toed footwear, hard hats, and safety eye wear in the event that a heavy particle or shower of smaller material unexpectedly falls. This is particularly important since the side walls are typically loose and covered with some fall-out material requiring hand work (accomplished as excavation proceeds) to obtain a view of representative in situ material. If after-compaction gradation samples are to be taken from the side walls, the non-representative fall-out material must be excluded from the sample. The use of lift-surface markers such as lime or plastic sheeting (as previously discussed) are important to permit the pit or trench inspectors to clearly distinguish one lift from another. Items of interest when inspecting a pit or trench include: depth of fines on the lift surfaces, distribution of fines within the lift, the overall appearance of each lift such as the segregation pattern, occurrence of voids, and the

Figure 11-6. Inspection trench through the New Melones Dam test fill

general "tightness" of the fill, including the stability of the side walls. Thorough documentation of the inspection trench is very important including photographs and a written record. It should be considered imperative that design personnel take part in the inspections.

Chapter 12
Evaluation of Test Fill Results

12-1. General

It has been shown in the previous portions of this manual that information accumulated from a test fill program consists of both qualitative and quantitative data. In analyzing that data and drawing conclusions for design purposes, it is necessary to consider all the data in that qualitative data should not be neglected or ignored even in the presence of strong quantitative data. This is true because quantitative data previously addressed is subject to considerable variation with location in a rockfill, may not be completely representative, or may have been obtained by methods which are not standardized. The remainder of this chapter will be directed at describing the typical usages of gathered data in assessing the placement and compaction procedures.

12-2. Settlement Data

Settlement data have generally been the most useful information for determining the best combination of loose-lift thickness, number of passes, roller type (compaction effort), and material gradation. Settlement data are normally plotted with settlement as the ordinate (y-axis) and actual loose-lift thickness as the abscissa (x-axis) and for each two passes of the roller. Of course, settlement can also be plotted against any of the other listed variables. The type plot used will be dependent upon the variable to be evaluated, but the data should always be plotted in several ways since some relationships will be more apparent in one form of plot compared with others. The settlement readings may be expressed as the actual values in cm or inches or as a percentage of the loose-lift thickness. If, as usual, settlement is to be compared for different loose-lift thickness, it is preferable to express it as a percentage of the loose-lift thickness because that number represents a relative densification. Figures 12-1 through 12-4 show example plots of percent settlement versus roller passes. Figure 12-5 through 12-7 show example plots of percent settlement versus loose-lift thickness.

12-3. Roller Passes

In evaluating the number of roller passes, economics must be considered in that a true optimum based on performance alone can rarely be selected. Instead, consideration must be given to the relative amount of additional settlement or compaction gained for additional passes as indicated by the slope of the settlement versus number of passes curve. Figures 12-1 through 12-4 show that in most (but not all) instances the curves tend to decrease in slope with number of passes even though the decrease may be relatively subtle. These figures also offer some comparative cases among different types of rollers. In addition, visual observations of the inspection trench and results of in situ density and percolation tests must be factored in to make a judgment based on the highest return for the effort put forth and the actual needs of the embankment, i.e., height, seismic risk, etc. More compaction can almost always be obtained by more passes, but the price that must be paid to achieve it becomes increasingly high and perhaps unjustified.

12-4. Gradation Data

Gradation data are usually obtained on the rockfill material before spreading and compaction and after compaction. Intermediate gradations have also been taken such as after certain numbers of passes of the roller or even after spreading if alternate spreading operations are under consideration to reduce material breakdown such as between rubber-tired equipment and the crawler tractor. Figures 12-8 through 12-12 show typical before- and after-compaction gradation curves. These figures were also selected because they compare vibratory rollers with 45-Mg (50-ton) rubber-tired rollers.

12-5. In Situ Density Data

In considering in situ density data, it is well to bear in mind the previous discussions concerning the major source of the shear strength between a sound rockfill and that which contains considerable fines. For sound rock, the key to its strength is interlocking among the larger particles which are segregated toward the bottom of the lift even if fines are generated on the surface of the lift. For materials containing appreciable fines mixed throughout the material such that the larger rocks tend to be separated and "float" within the finer fraction, the shear strength and compressibility will reflect the density of the mass much more directly. For fill composed of sound rock, in situ density numbers are of lesser interest than the observation that the lifts are expectably segregated and stratified with interlocking of the larger particles. For "dirtier" rockfill where the fines will control the strength and compressibility, variability of in situ density with type of roller, number of passes, loose-lift thickness, and use of water is of much greater importance. The Seven Oaks Dam test fill program included an unusually large number of 1.2-m (4-ft) diameter, water-volume, ring density tests.
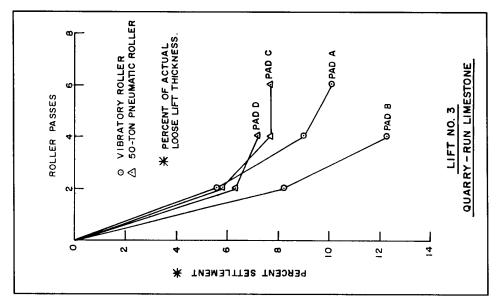
Figure 12-2. Percent settlement versus roller passes, lift 3, test fill 4A, Cerrillos Dam
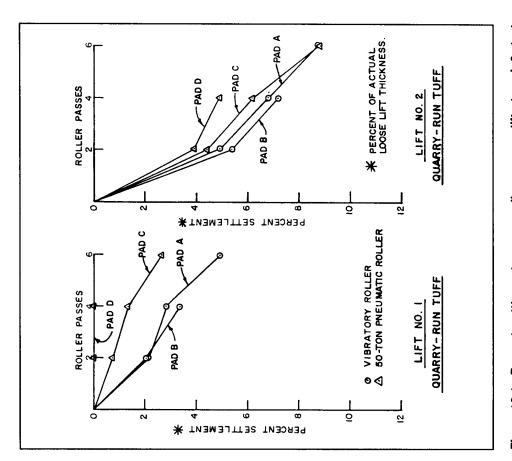

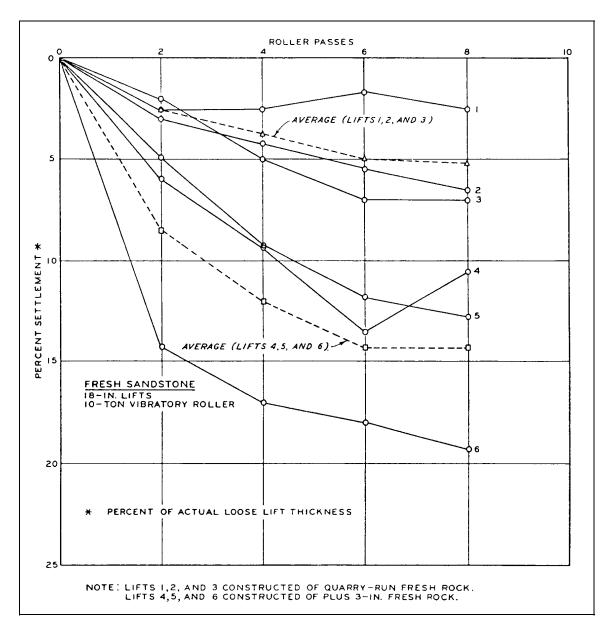**Figure 12-1. Percent settlement versus roller passes, lifts 1 and 2, test fill 4A, Cerrillos Dam**

ROLLER PASSES

PERCENT SETTLEMENT *

AVERAGE (LIFTS 1,2, AND 3)

AVERAGE (LIFTS 4,5, AND 6)

FRESH SANDSTONE
18-IN. LIFTS
10-TON VIBRATORY ROLLER

* PERCENT OF ACTUAL LOOSE LIFT THICKNESS

NOTE: LIFTS 1,2, AND 3 CONSTRUCTED OF QUARRY-RUN FRESH ROCK.
LIFTS 4,5, AND 6 CONSTRUCTED OF PLUS 3-IN. FRESH ROCK.

Figure 12-3. Percent settlement versus roller passes, Gilham Dam test fill

The portion of these data given in Figures 12-13 through 12-18 show in situ densities achieved by different rollers for a 45.7-cm (18-in.) lift thickness over a range in number of passes for the variety of rock types involved. Figures 12-13 through 12-15 show the actual results of the in situ density tests. The range of densities indicated are typical of those seen for rockfill materials. Because of the variation in the gradations of the full-scale material derived from the density tests including a range in maximum particle size, the Los Angeles District personnel astutely realized that direct comparison of the density values of Figures 12-13 through 12-15 was not valid. To compare the performance of the rollers on a common basis, it was decided to correct the fill density test values to those corresponding to the minus 7.6-cm (3-in.) and minus 3.8-cm (1.5-in.) fractions of each fill density sample. The correction was by use of the equation given in EM 1110-2-1911, Appendix B (see paragraph 11-2c, equation 11-1) where the gradation curve for each full-scale density sample was used to obtain the percent coarse fraction and percent fine fraction. In correcting the total material density to that of the minus 7.6-cm (3-in.) fraction within it, the percent coarse fraction "c" was the percent retained on the 7.6-cm (3-in.) sieve and the percent fine fraction "f" was then 1-c. Likewise, to correct
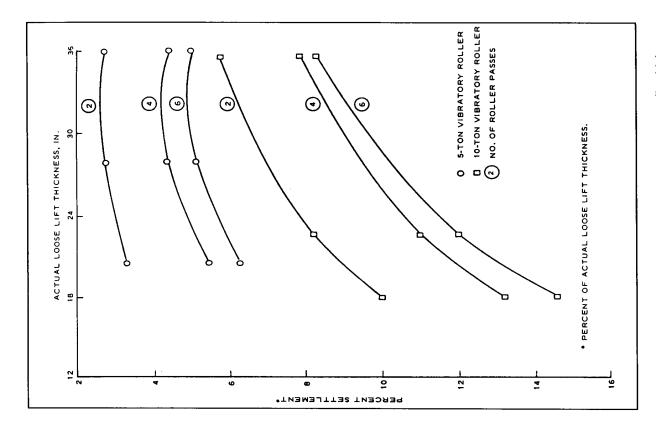
Figure 12-5. Percent settlement versus actual loose-fit thickness, test fill series 2, Cougar Dam
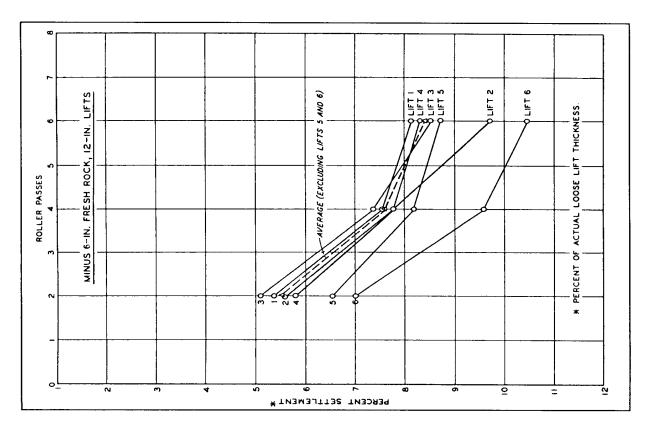
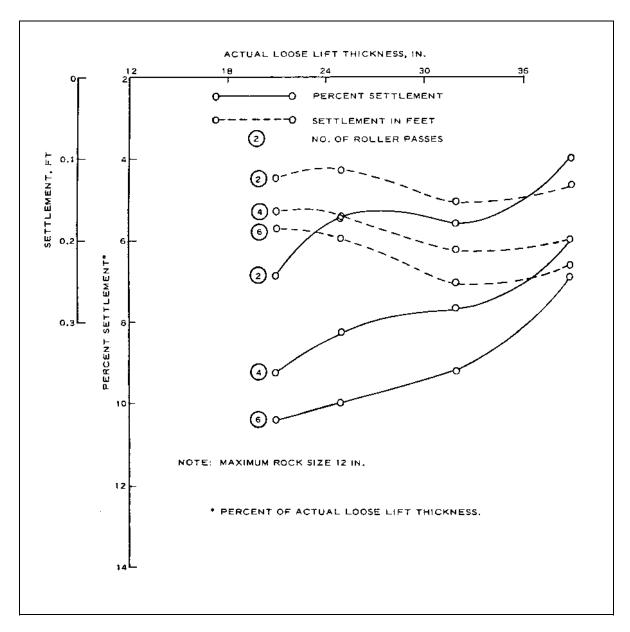Figure 12-4. Percent settlement versus roller passes, test fill 3, New Melones Dam

Figure 12-6. Percent settlement versus actual loose-fit thickness, test fill series 3, Cougar Dam

the total material density to that of the minus 3.8-cm (1.5-in.) fraction within it, the percent coarse fraction "c" was that retained on the 3.8-cm (1.5-in.) sieve. Figures 12-16 through 12-18 show the data of Figures 12-13 through 12-15 corrected to the densities of the minus 7.6-cm (3-in.) fractions. It is seen that the 45.3-Mg (50-ton) rubber-tired roller performed about as well as the vibratory rollers. As a matter of interest, the recommended compaction procedure for the rockfill of Seven Oaks Dam shell became a 45.7-cm (18-in.) loose lift thickness and 6 passes of the 9.1-Mg (10-ton) vibratory roller. For "dirtier" rockfill of the same parentage as the

shell materials, a rockfill transition zone was provided downstream of the core of the dam, and its recommended compaction was 30.5-cm (12-in.) loose lifts and 4 passes of the 9.1-Mg (10-ton) vibratory roller.

12-6. Inspection Trench

Figures 12-19 through 12-22 are provided as examples of inspection trench observations. Figure 12-19 shows the sound rockfill for New Melones Dam where the important interlocking of the larger particles is evident with
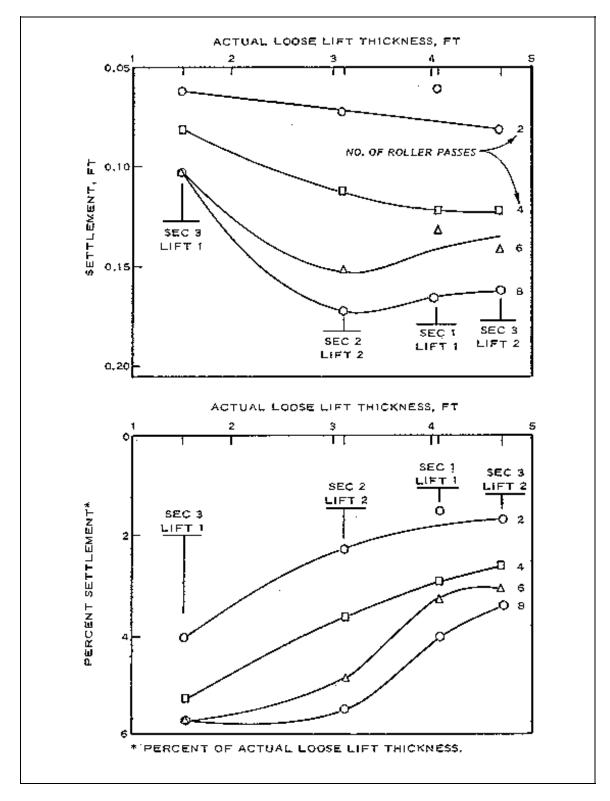
Figure 12-7. Percent settlement versus actual loose-fit thickness, Laurel Dam test fill
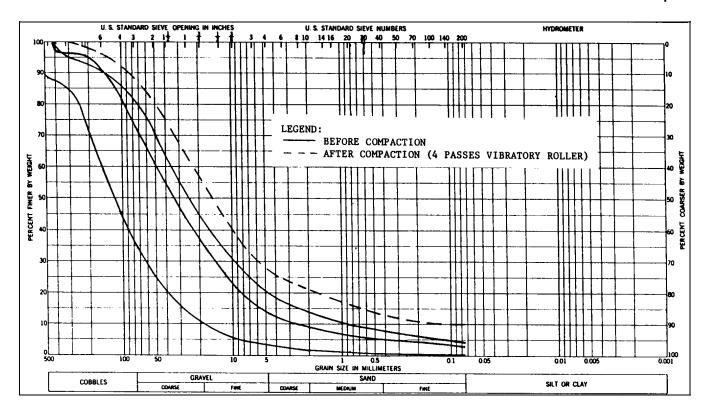
Figure 12-8. Before and after-compaction gradations after 4 passes of the vibratory roller, quarry-run siltstone, Cerrillos Dam
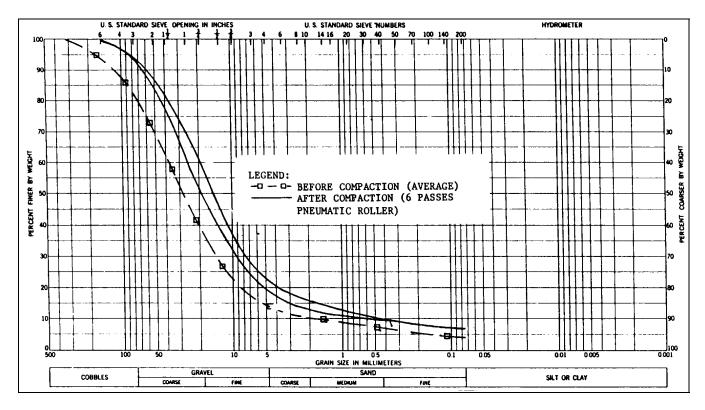
Figure 12-9. Before and after-compaction gradations after 6 passes of the pneumatic roller, quarry-run siltstone, Cerrillos Dam
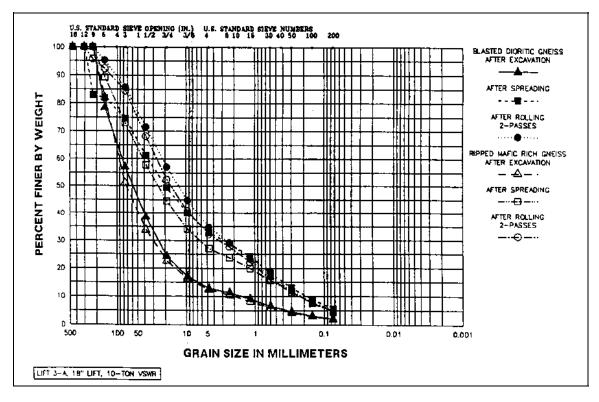
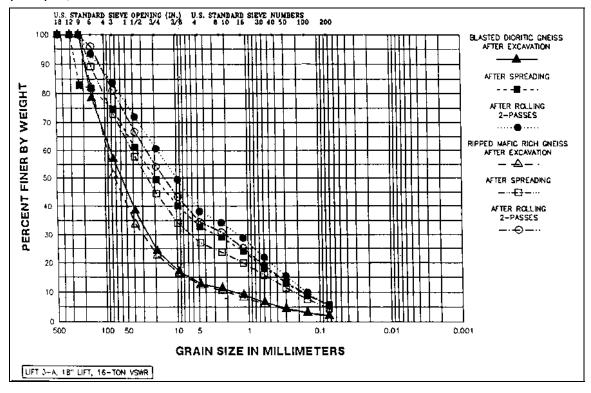**Figure 12-10. Change in gradation after 2 passes of 9.1-metric ton (10-ton) vibratory roller, 45.7-cm (18-in.) lift, Seven Oaks Dam materials**

Figure 12-11. Change in gradation after 2 passes of 14.5-metric ton (16-ton) vibratory roller, 45.7-cm (18-in.) lift, Seven Oaks Dam materials
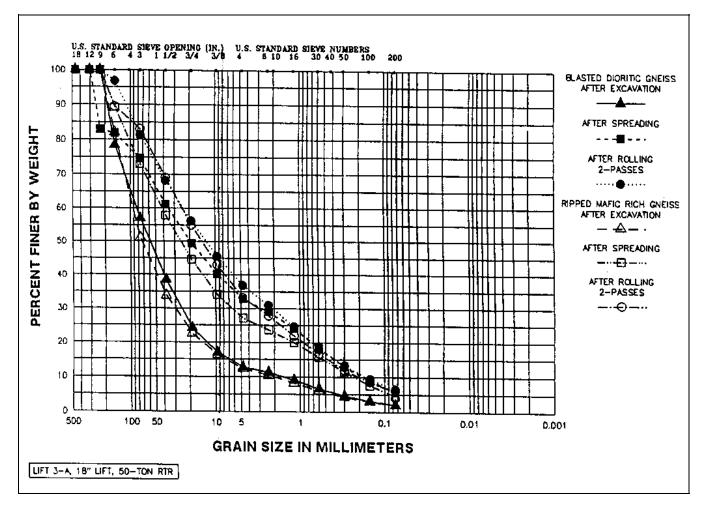
Figure 12-12. Change in gradation after 2 passes of 45.4-metric ton (50-ton) rubber-tired roller, 45.7 cm (18-in.) lift, Seven Oaks Dam materials

relatively little fines distributed throughout. Figure 12-20 shows a New Melones material with considerable fines where the overall uniform density with no significant segregation is apparent. It is also noted from the lower photograph of Figure 12-20 that the lifts are very nicely bonded as indicated by no indication of lift boundary other than the lime lift marker material (very thin, horizontal white seam across the photo). Figures 12-21 and 12-22 are from the test fills for Cerrillos Dam and also show the relative compactness and uniformity of that material containing appreciable fines.

12-7. Test Fill Report

After completion of the test fill program, a comprehensive formal report should be prepared as a project Design Memorandum in its own right or as a major portion of a Design Memorandum. Where a test quarry program was also performed in conjunction with the test fill program, the comprehensive report should treat both subjects. A typical test quarry and test fill report outline is given in Table 12-1. The test quarry portion of Table 12-1 was previously suggested in Table 7-1.
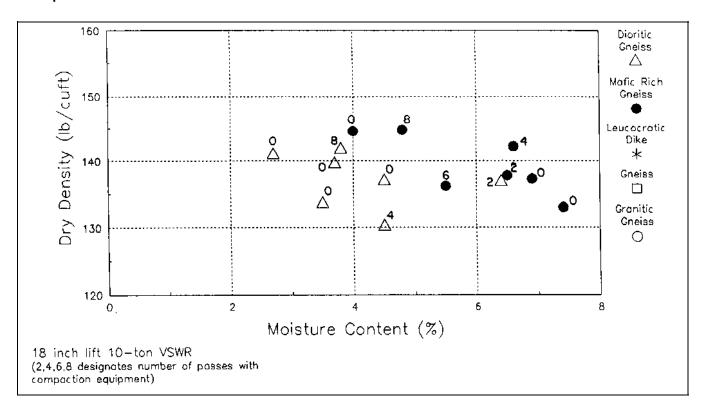
Figure 12-13. In situ density tests, 45.7-cm (18-in.) lift, 9.1-metric ton (10-ton) vibratory roller, Seven Oaks Dam materials
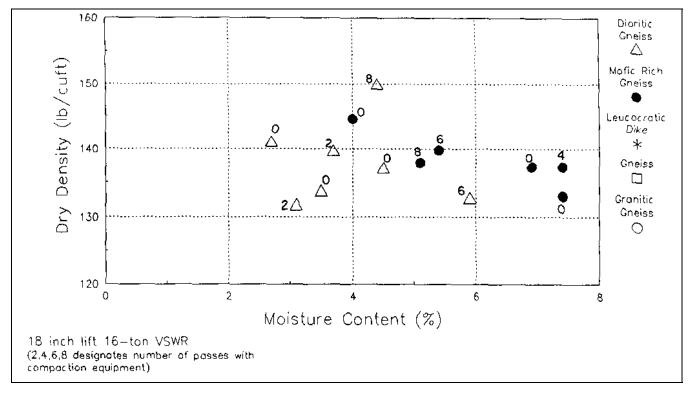

Figure 12-14. In situ density tests, 45.7-cm (18-in.) lift, 14.5-metric ton (16-ton) vibratory roller, Seven Oaks Dam materials
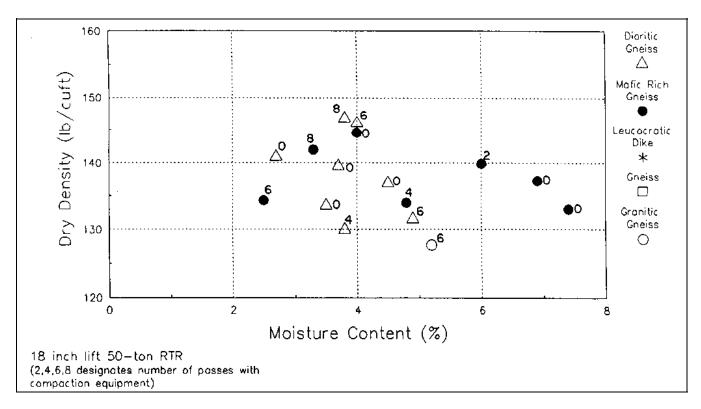
Figure 12-15. In situ density tests, 45.7-cm (18-in.) lift, 45.4-metric ton (50-ton) rubber-tired roller, Seven Oaks Dam materials
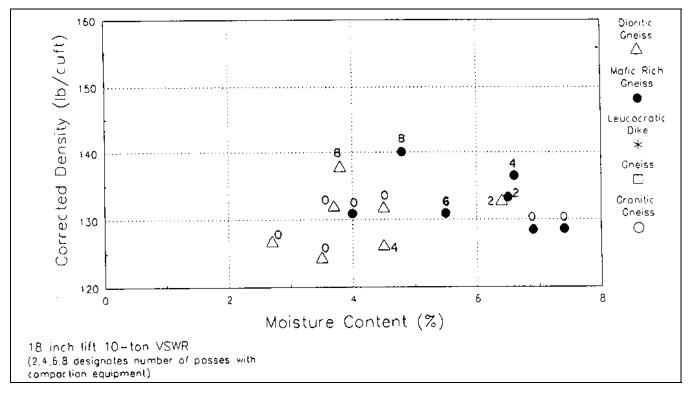

Figure 12-16. Corrected in situ densities, 45.7-cm (18-in.) lift, 9.1-metric ton (10-ton) vibratory roller, Seven Oaks Dam materials
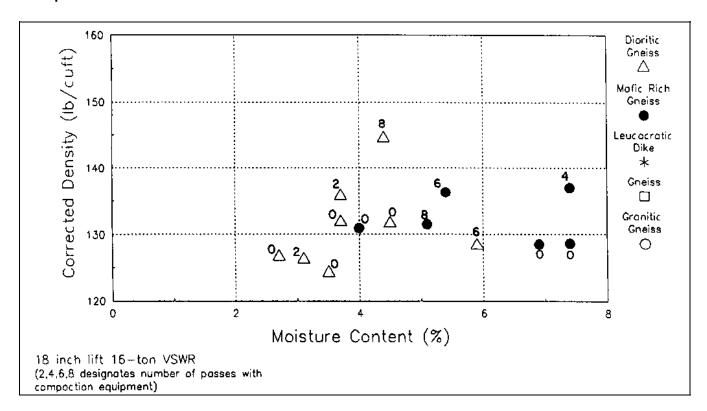
Figure 12-17. Corrected in situ densities, 45.7-cm (18-in.) lift, 14.5-metric ton (16-ton) vibratory roller, Seven Oaks Dam materials
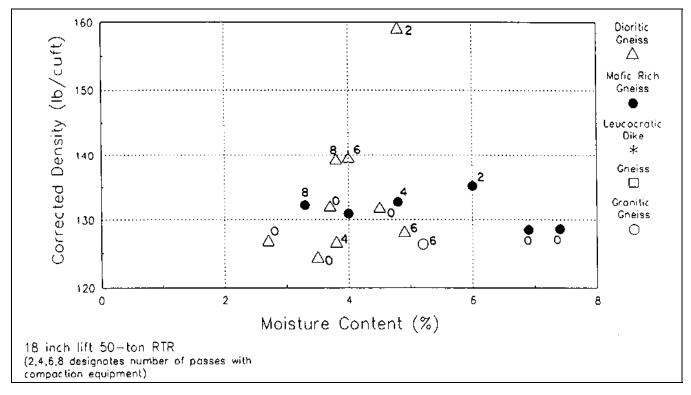

Figure 12-18. Corrected in situ densities, 45.7-cm (18-in.) lift, 45.4-metric ton (50-ton) rubber-tired roller, Seven Oaks Dam materials
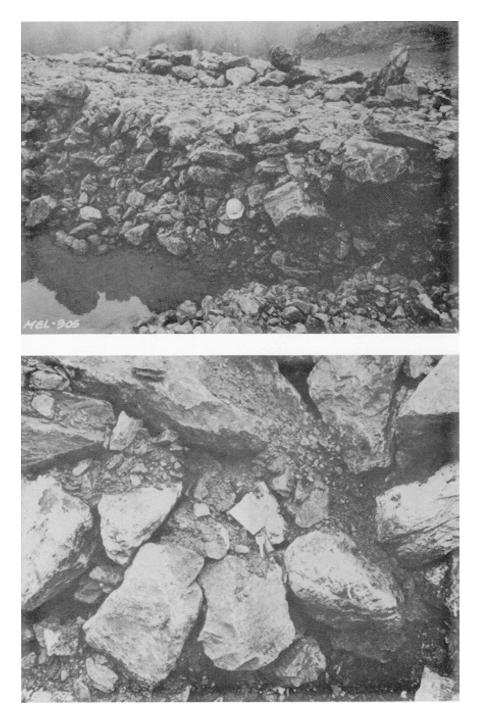
Figure 12-19. Photographs of sound rockfill, New Melones Dam test fill inspection trench

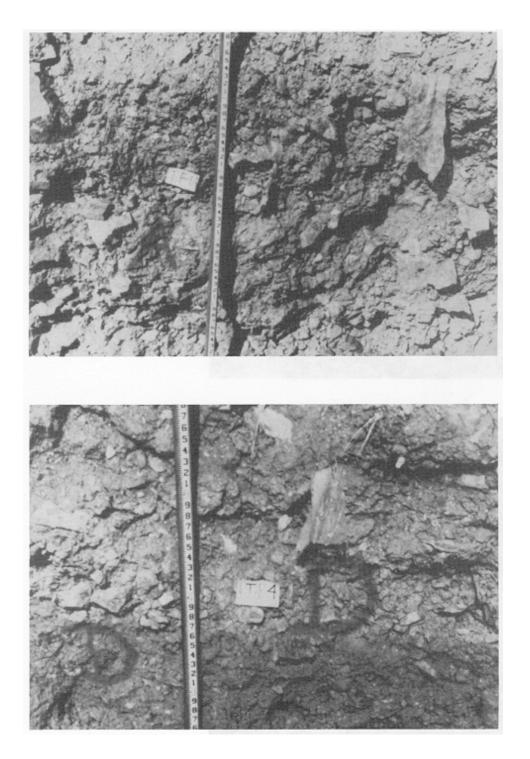Figure 12-20. Photographs of rockfill containing appreciable fines, New Melones Dam test fill inspection trench

Figure 12-21. Photographs of rockfill containing appreciable fines, Cerrillos Dam test fill No. 4 inspection trench
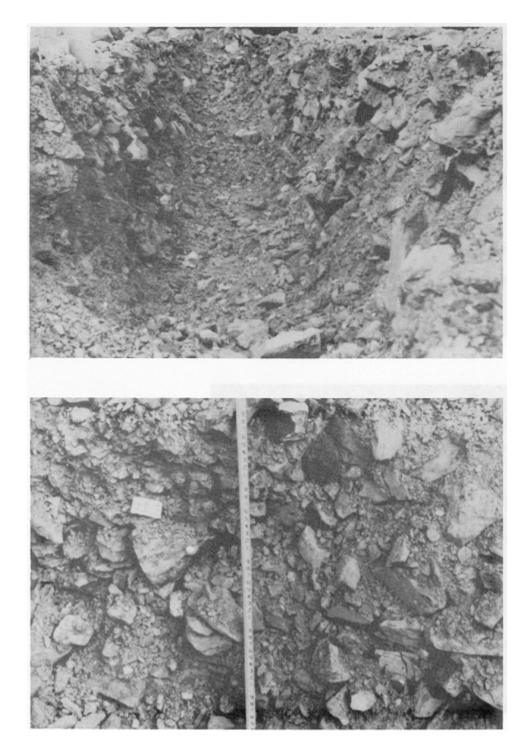
Figure 12-22. Photographs of rockfill containing appreciable fines, Cerrillos Dam test fill No. 5 inspection trench

Table 12-1
Typical Test Quarry and Test Fill Report Outline

1. Executive Summary

2. Table of Contents

3. Section 1 - Introduction

   a. Authorization
   b. Scope and purpose
   c. Location of the project

4. Section 2 - Site Geology

   a. Topography
   b. Geology

5. Section 3 - Project Plan

   a. Project features utilizing rockfill
   b. Expected borrow sources and locations

6. Section 4 - Site Selection for Test Quarry and Test Fill

   a. General discussion of site selection factors and judgments including borehole information
   b. Selected locations of test quarry and test fill
   c. General preliminary site preparation such as removal of and destination of overburden, access road aspects, etc.

7. Section 5 - Test Quarry

   a. Test quarry design and objectives

      1) Discussion of objectives
      2) Overview of site selection criteria
      3) Thorough presentation of design including layout and slope stability

   b. Geological conditions in the test quarry
   c. Description of each test blast

      1) Rock type and condition
      2) Hole pattern
      3) Delay pattern
      4) Hole depths and loading design
      5) Explosives, detonators, and delays
      6) Blasted rock mass description
      7) Quarry-run gradation
      8) Laboratory tests and results
      9) Conclusions

   d. Drilling, loading, and hauling equipment and procedures
   e. Description of the results of each presplit slope blast

      1) Rock type and condition
      2) Presplit hole and explosive charge configuration
      3) Presplit slope condition
      4) Rock joint analysis and slope stability
      5) Conclusions

(Continued)

Table 12-1 (Concluded)

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   f. Rock processing results

   1) Description of processing objective
   2) Description of rock processing equipment
   3) Results of processing each rock type and condition
   4) Gradations and particle shapes
   5) Degradation during each stage of processing
   6) Laboratory tests and results

   g. Conclusions and recommendations

8. Section 6 - Test Fill

   a. Description of the test fill program including materials, layout, and compaction equipment to be assessed
   b. Description of tests and measurements to be performed

   1) Description of test procedures and schedule of tests
   2) Locations including settlement grid layout

   c. Foundation preparation

   1) Description of the foundation
   2) Treatment prior to leveling pad construction (if any) such as special rolling to reduce compressibility
   3) Materials and construction of leveling pad, including equipment types
   4) Justification for, locations of, descriptions of, and installation procedures for settlement plates

   d. Construction
   e. Field tests and measurements, i.e., in situ densities, gradations, etc.
   f. Laboratory tests and results (if any)
   g. Analysis and discussion of data

9. Section 7 - Inspection Pits or Trenches

   a. Description of inspection pits or trenches and locations, method of excavation
   b. Discussion of observations and any sampling

10. Section 8 - Conclusions and Recommendations

   a. Conclusions including lessons learned
   b. Recommendations

APPENDICES -- Laboratory Test Sheets, Boring Logs, Field Gradation Test Results, Description of Rock Processing Equipment, Photographic Documentation of All Aspects of Test Quarry and Test Fill, etc.